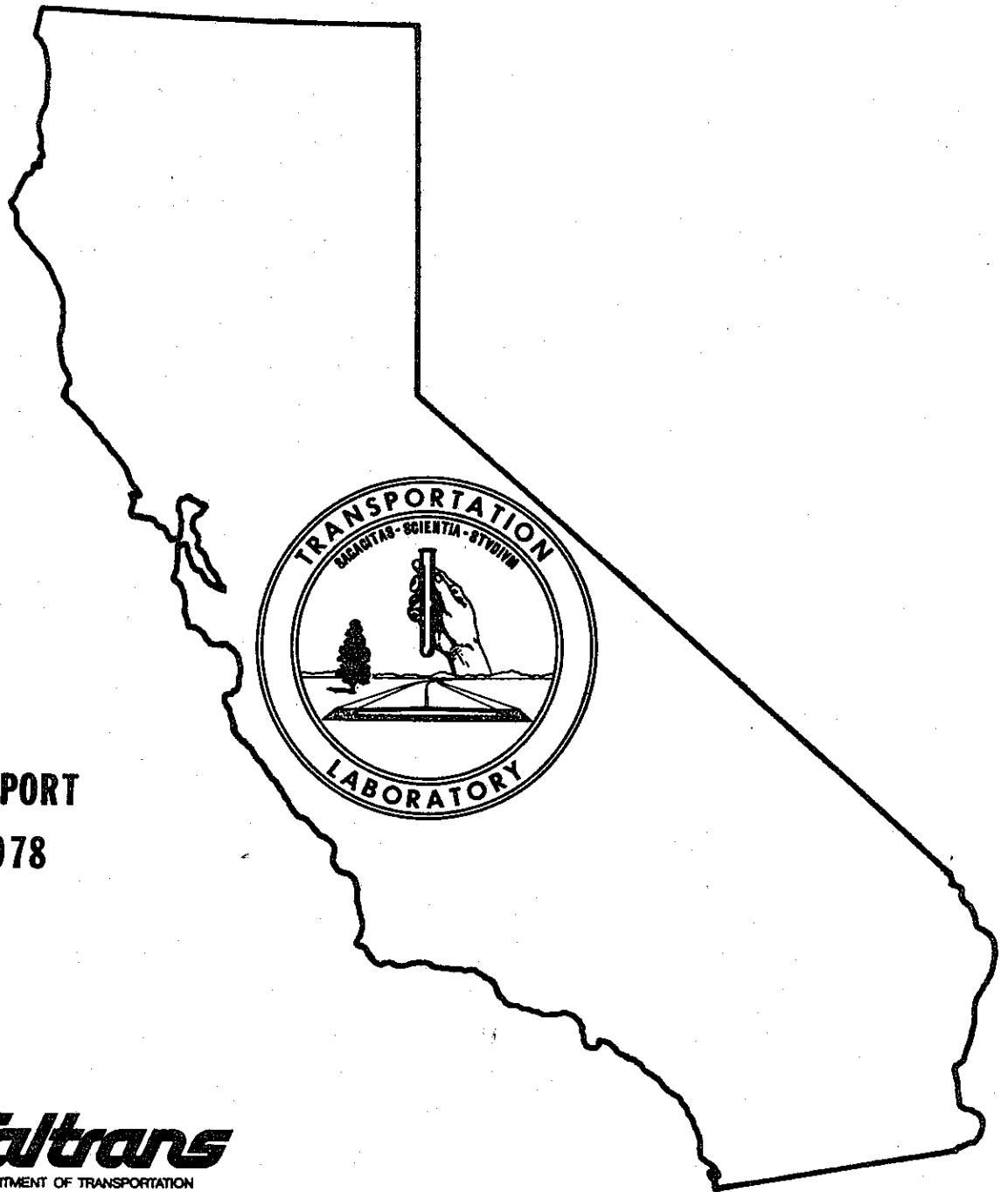


# A REVIEW OF CALIFORNIA STRUCTURAL SECTION DESIGN PROCEDURES



**FINAL REPORT**  
**APRIL 1978**

**Caltrans**  
CALIFORNIA DEPARTMENT OF TRANSPORTATION

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1. REPORT NO. FHWA-CA-TL-78-07		2. GOVERNMENT ACCESSION NO.		3. RECIPIENT'S CATALOG NO.	
4. TITLE AND SUBTITLE A REVIEW OF CALIFORNIA STRUCTURAL SECTION DESIGN PROCEDURES				5. REPORT DATE April 1978	
				6. PERFORMING ORGANIZATION CODE	
7. AUTHOR(S) Karl L. Baumeister and Robert N. Doty				8. PERFORMING ORGANIZATION REPORT NO. 19301-633115	
9. PERFORMING ORGANIZATION NAME AND ADDRESS Office of Transportation Laboratory California Department of Transportation Sacramento, California 95819				10. WORK UNIT NO.	
				11. CONTRACT OR GRANT NO. D-5-41	
12. SPONSORING AGENCY NAME AND ADDRESS California Department of Transportation Sacramento, California 95807				13. TYPE OF REPORT & PERIOD COVERED Final	
				14. SPONSORING AGENCY CODE	
15. SUPPLEMENTARY NOTES This study was conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration.					
16. ABSTRACT The structural section design procedures in use by the California Department of Transportation (Caltrans) are generally empirical in nature. These procedures are based upon previously encountered general design conditions such as traffic, materials, and environment but are not responsive to changes in these conditions. In an attempt to correct some of these deficiencies, several "rational" structural section design methods were reviewed. A summary of this review is presented and some modifications of the current Caltrans' procedures are suggested in an attempt to make these procedures more responsive to the variety of design considerations encountered in California. The effect of these suggested changes has been estimated using the actual performance of several pavements wherein the as-built design was compared with designs developed per the present Caltrans' procedure and per some of these "rational" procedures.					
17. KEY WORDS Portland cement concrete pavements, asphalt pavements, pavement design, elastic theory.			18. DISTRIBUTION STATEMENT No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161		
19. SECURITY CLASSIF. (OF THIS REPORT) Unclassified		20. SECURITY CLASSIF. (OF THIS PAGE) Unclassified		21. NO. OF PAGES 70	
				22. PRICE	



STATE OF CALIFORNIA  
DEPARTMENT OF TRANSPORTATION  
DIVISION OF CONSTRUCTION  
OFFICE OF TRANSPORTATION LABORATORY

April 1978

FHWA No. D-5-41  
TL No. 633115

Mr. C. E. Forbes  
Chief Engineer

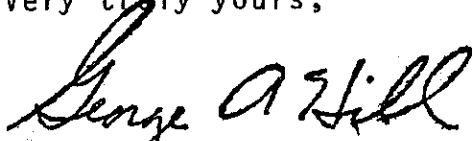
Dear Sir:

I have approved and now submit for your information this  
final research project report titled:

A REVIEW OF CALIFORNIA STRUCTURAL  
SECTION DESIGN PROCEDURES

Study made by ..... Roadbed and Concrete Branch  
Under the Supervision of ..... Donald L. Spellman, P. E.  
Principal Investigator ..... Robert N. Doty, P. E.  
Co-Investigator ..... Karl L. Baumeister, P. E.  
Report Prepared by ..... Karl L. Baumeister  
and  
Robert N. Doty

Very truly yours,



GEORGE A. HILL  
Chief, Office of Transportation Laboratory

Attachment  
KLB:bjs



## ACKNOWLEDGEMENTS

This study, which was titled "Revision of the California Structural Section Design Method", was conducted by the Office of Transportation Laboratory, Division of Construction of the California Department of Transportation (Caltrans). The author wishes to extend his sincere appreciation to the Caltrans' District and Translab employees who assisted in the collection and processing of the data accumulated for this study and especially to Robert E. Smith, formerly with Caltrans, who was instrumental in initiating this project. In addition, appreciation is extended to Ms. Betty Stoker and Eileen Howe for their typing and editorial assistance.



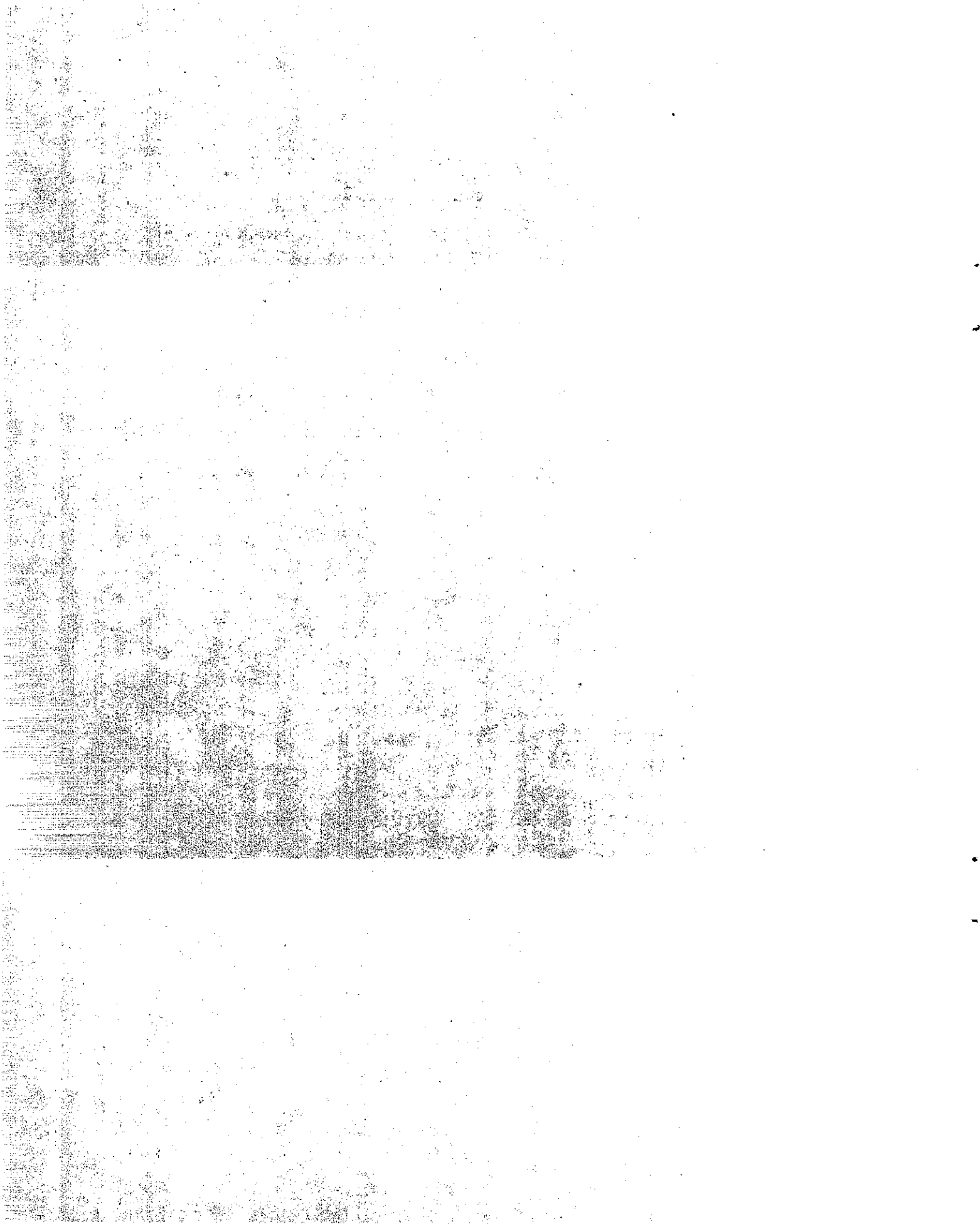
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## I. INTRODUCTION

The flexible pavement structural section design method used by the California Department of Transportation, which applies to asphalt concrete (AC) over untreated aggregate base (AB) or cement treated base (CTB), is largely an empirical method based on pavement performance in California and road performance tests such as the AASHO Road Test in Illinois. The original version of this design method was introduced over 25 years ago. However, materials specifications, construction practices and equipment, and design constants have undergone appreciable modification since then. Practically all these modifications resulted in increases in structural section design thicknesses. The net effect of all the changes has not been fully evaluated for pavements constructed after 1963, by which time most of the major changes had been incorporated into the design method.

The selection of materials for roadways is dependent on factors that are continually changing. For example, there are changes in the composition of optimum structural sections in any given location because of changes in costs of component materials and construction methods. Also, the materials properties on which the present design method is based are tied to relatively fixed specification limits, testing procedures, and contractor operations. In addition, there is a wide variation in the climatic conditions found within California. Because the resilience (elastic deformation) of AC varies greatly with temperature (as well as with duration of loading), these climatic differences influence the performance of AC. Also, the resilience of subgrades and untreated bases varies with moisture content, another in-situ variation not presently accounted for by the Caltrans' procedure. Even though the resilience of the AC, base and subgrade can combine to substantially influence the fatigue life of the roadway, the structural section design method does not, at present, consider the combined effect of climatic conditions on these variables. Consequently, the

objective of this research was to produce a flexible pavement design framework capable of accounting for these variables. It was assumed that this framework would incorporate both theoretical and empirical approaches in order to properly account for the differences in materials, construction, and climatic conditions encountered in California while continuing to apply the knowledge gained during the use of the current method.

The most frequently mentioned theoretical structural section design approach for flexible pavement incorporates elastic layered theory. Thus, this study included a comparison of this approach and the current Caltrans' procedure for the design of AC over untreated aggregate base. In the design of structural sections containing AC and cement treated or lean concrete (rigid) bases, however, the elastic layer method has its limitations. Studies reported by others indicate that these rigid bases are better considered as being of limited size because of shrinkage cracks and the problem of edge loading. Thus, this study included a comparison of the present structural section design method for AC over CTB with a Westergaard-based method which accounts for the concentration of stress which occurs when loads are applied near the edge of rigid slabs of limited size while also incorporating elastic layer assumptions.

The present Caltrans' method of rigid pavement design, which consists of a modified version of the PCA design procedure, generally provides designs that achieve the design life for the "truck" lane but considerably greater service lives for passing lanes. A possible explanation for this appeared to be associated with the method used to characterize the heaviest and lightest loads and the inability of the existing design method to consider the duration of rest periods between wheel loads. Consequently, the use of a load safety factor that increases with traffic

density was explored. This approach was based on data from the AASHO Road Test and modifications thereof to correlate with pavement performance..

## II. FINDINGS AND CONCLUSIONS

Comparison of "as-built" structural sections in several California locations with "designs" for the same locations based upon current Caltrans' procedures and based upon possible alternate theoretical design procedures revealed the following:

### Flexible Pavement Design

1. The theoretical flexible pavement design procedure examined provides results that are different from the current Caltrans' procedure in most cases;
2. Both the current and the theoretical procedures provide AC designs that are conservative and, therefore, appear to provide a factor of safety based on the actual performance of a selected group of pavements that have experienced 7 to 26 years of service prior to resurfacing. These conservative designs are desirable because past experience indicates that traffic often increases both in weight and volume at a rate greater than that anticipated.
3. Both the current and the theoretical procedures need modification to adequately account for climatic extremes found in California;
4. When using the current Caltrans' (gravel equivalent) design method, the structural advantage of increasing untreated base thickness may diminish as the base thickness increases, and
5. The theoretical procedure described herein should not be adopted as a replacement for the current Caltrans' flexible pavement design procedure. It can, however, be used to analyze the effect of proposed overloads on existing structural sections.

## Semi-Rigid Pavement

1. Use of the current Caltrans' procedure results in somewhat conservative cement treated base (CTB) thicknesses beneath AC for pavements constructed prior to the load limit increase adopted in 1975;
2. A procedure implementing the Westergaard edge loading design criteria provides CTB thicknesses slightly conservative as related to the performance of twenty-seven in-service roadways but generally less than those dictated by the current Caltrans' procedure;
3. The Westergaard procedure is more sensitive to load magnitude than the current Caltrans' procedure;
4. Use of the Westergaard procedure results in CTB thicknesses greater than those obtained using the PCA design method. However, this additional thickness appears to be required for satisfactory roadway performance;
5. Based upon analyses using the Westergaard approach, the substitution of lean concrete base (LCB) for CTB will permit a theoretical reduction of 10% in the thickness of the base;
6. The use of a single equivalency factor (gravel factor, or  $G_f$ ) regardless of the strength of the base material and stress and strain within the base material is not prudent, and
7. The Westergaard design scheme referred to in this report should be used to develop alternate designs on two or three projects and portions of each job constructed in accordance with the Westergaard and the current Caltrans' procedure for comparative evaluation if the anticipated amount of future use of semi-rigid pavement so warrants. These projects should include at least

one for which the Westergaard approach is more conservative than the current Caltrans' procedure and one project for which the Westergaard method provides a less conservative design.

### Rigid Pavement Design

1. For conditions typical of the AASHO test road and many areas within California, the AASHTO and Texas design procedures dictate greater PCC pavement thicknesses than does the current Caltrans' procedure for heavy traffic (high T.I.);
2. Load safety factors (LSF's) should be selected based upon the magnitude (T.I.) of the anticipated traffic in the truck lane(s), and
3. Considering the actual performance of PCC pavement, the design period of 20 years now being used by Caltrans for rigid pavement design should be reviewed and, if appropriate, increased.

### III. IMPLEMENTATION

1. The theoretical flexible pavement design procedure examined should not be adopted as a replacement for the current Caltrans' design procedure. Its continued use is appropriate, however, when analyzing the effect of proposed overloads on existing structural sections as the current Caltrans' design procedure cannot be used for this type of problem.

2. The Westergaard method of semi-rigid pavement design should be used on a trial basis on portions of two or three contracts if the anticipated volume of future semi-rigid pavement construction so warrants. When lean concrete base (LCB) of the quality currently being obtained is used in lieu of CTB for semi-rigid pavement, a 10% reduction in the thickness of the base should be permitted.

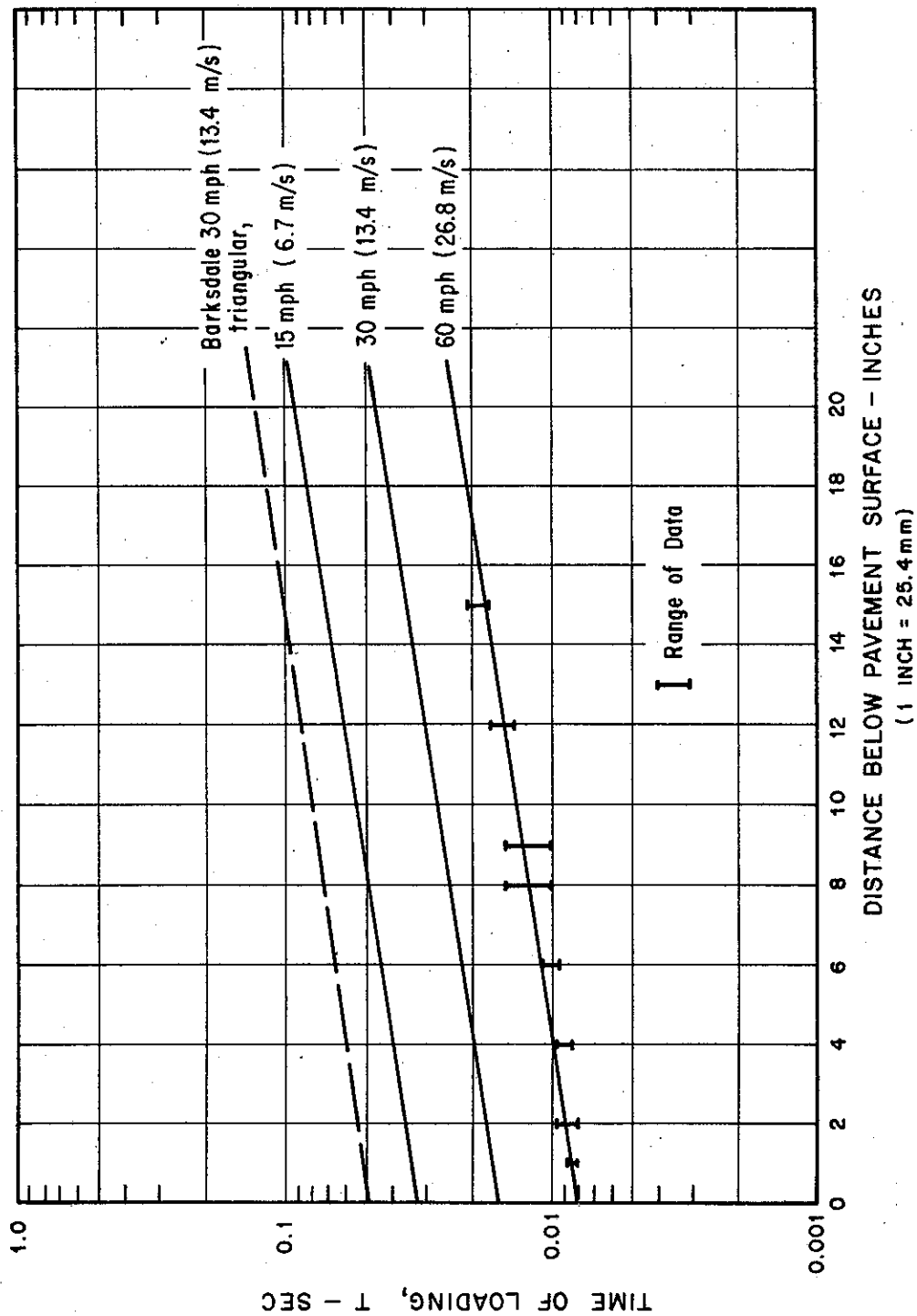
3. The relationship of load safety factor (LSF) and traffic intensity, as indicated by the design T.I., should be implemented for the design of PCC pavement. Also, design periods in excess of twenty years should be used whenever possible for PCC pavements.

#### IV. PROCEDURE AND DISCUSSION

##### A. Flexible Pavement Design

A "rational" computerized approach to structural section design developed at the University of California, Berkeley, and designated as PSAD-2, was examined as a possible modification to, or replacement for, the Caltrans' flexible pavement design procedure. This approach, which incorporates the CHEV 5L elastic layer procedure(1), appeared to be the most promising of the systems available. It was combined with strain-fatigue relationships developed by Santucci(2) in an attempt to incorporate climatic and other factors not specifically considered for flexible pavement design per the current Caltrans' design procedure. To evaluate this structural section design procedure, twenty-three existing structural sections were re-designed for the traffic they had been subjected to prior to the need for major rehabilitation and these theoretical structural sections then compared with the existing structural sections and with structural sections designed per the current Caltrans' design procedure and the AASHO 1972 Interim Guide procedure for these same traffic intensities.

The stiffness or "modulus of resilience" of AC is very sensitive to load duration and temperature so these two parameters were used in order to calculate the AC modulus of resilience using a computer program developed at the University of California, Berkeley (ENG 081). Regarding time of loading, McLean(3) conducted an extensive investigation into stress versus time-of-loading relationships based on (1) laboratory tests to simulate pavement loading conditions, (2) previous research by others, and (3) elastic layer analysis (CHEV 5L and PSAD-2). Figure 1 shows the relationships developed by McLean. The results of other studies by Brown(4) and Dempwolff and Sommer(5) were found to be in reasonable agreement with McLean's conclusions, in spite of some differences in approach. In addition, the results of these studies



DURATION OF LOADING IN RELATION TO  
DEPTH BELOW PAVEMENT SURFACE AND VEHICLE SPEED

FIGURE 1

indicate that there are some differences in the duration of the vertical and horizontal stresses at a given depth within the AC. However, McLean indicates that the vertical and horizontal stress duration is about equal at the bottom of the upper structural layer, which is where fatigue failure usually appears to start in flexible pavement(6). Consideration was also given to the effect of grade on vehicle speed per Figure 2, which was published in reference 7. For the purposes of this study, a 0.015 second load duration was selected for typical highway pavement based upon McLean's findings.

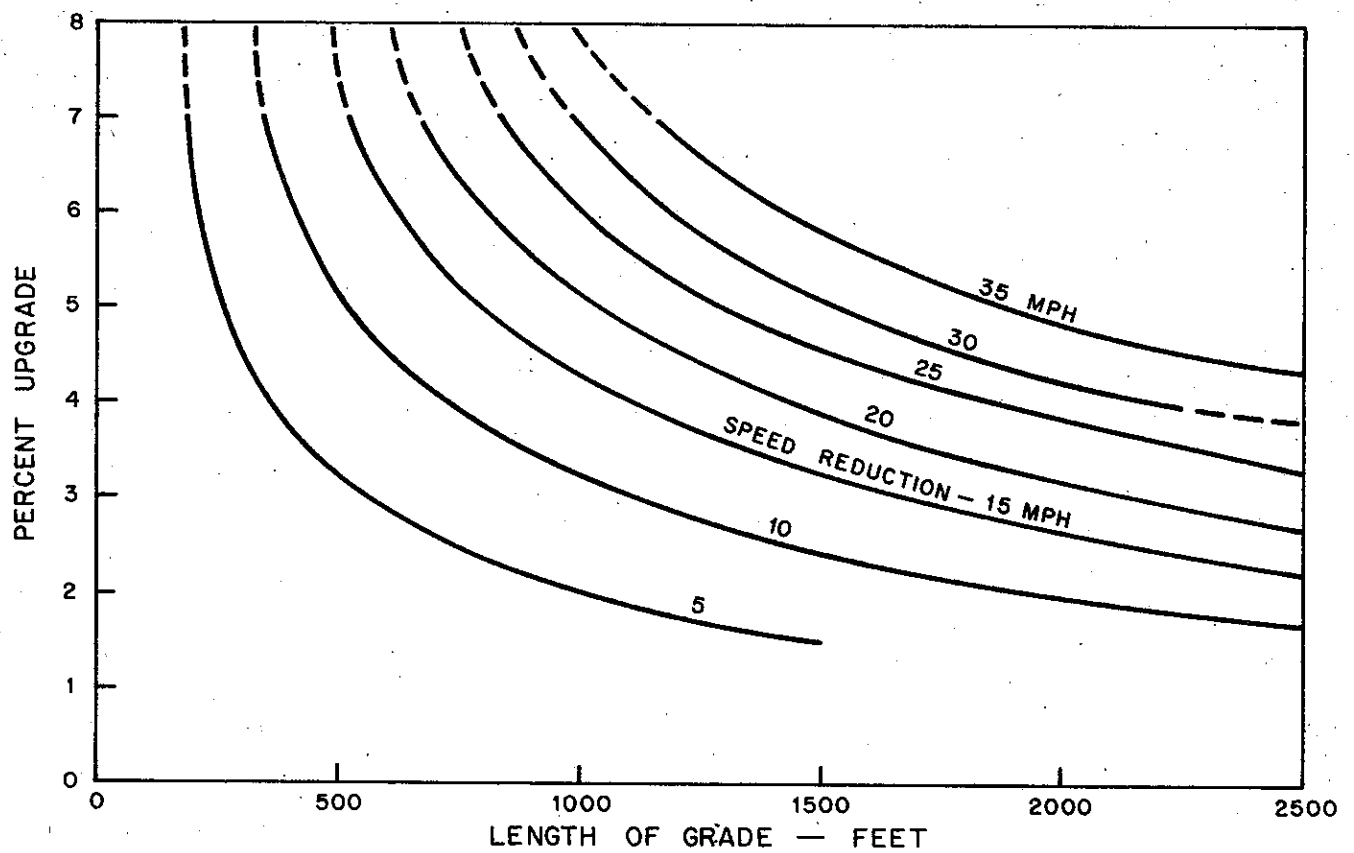
Conditions leading to fatigue failure in cold regions are considerably different than conditions in moderate climates, as indicated by studies in pavement near Regina, Canada. Bergan and Monismith(8), in a comparison of pavement performance using elastic layer theory, indicated that a load duration of 0.05 seconds gave theoretical answers compatible with experience when fatigue life was considered. This load duration was therefore used in analyzing two pavements in cold regions of California that had average annual temperatures of about 45°F (7°C).

Because the modulus of resilience of the AC will vary as the temperature varies, four moduli of resilience were determined for the AC in each location. To accomplish this, average monthly temperatures were determined for the winter and summer months. The remainder of the year was divided into two periods: March, April, and November; and May, September, and October, and average AC resilient moduli estimated for these four periods for each location. The asphalt and asphalt concrete were characterized as follows for these calculations:

Penetration of recovered asphalt = 35

Ring and ball softening point of recovered asphalt = 129°F (54°C)

Void content of the asphalt concrete = 5%



ASSUMED TYPICAL HEAVY TRUCK OF 400 POUNDS PER HORSEPOWER

CRITICAL LENGTHS OF GRADE FOR DESIGN - AASHTO POLICY

FIGURE 2

Asphalt content of the asphalt concrete by  
weight of the dry aggregate = 5.25%

Asphalt volume by total volume of the asphalt concrete = 11%

Specific gravity of the asphalt concrete aggregate = 2.65

To estimate a subgrade modulus of resilience for each location, the correlation between R-value and modulus of resilience used by Santucci(2) was employed for R-values of 10 or greater (see Figure 3). For R-values between 5 and 11, the equation  $E_s = 1300 + 140R$  was used. This is based on a correlation established by the California Department of Transportation for CBR versus R-value combined with the commonly used equation  $E_s = 1500 \times \text{CBR}$ . The 20th percentile R-value determined for 300 psi (2068 kPa) exudation pressure was used for this calculation. It was necessary to adjust 400 psi (2757 kPa) R-values to 300 psi (2068 kPa) R-values in most cases. This was done per Reference 9.

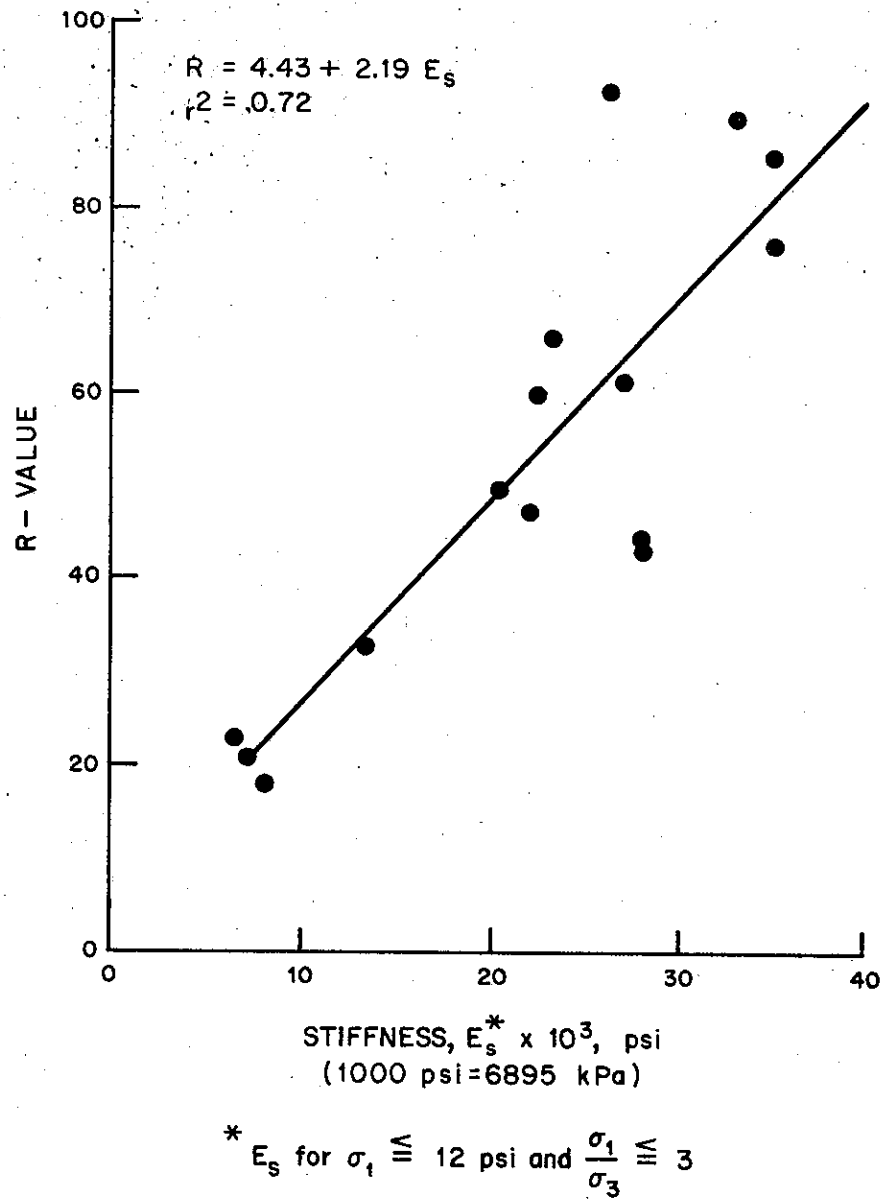
Investigations by Dehlen(10), Hicks(11), and Monismith(12) have revealed that the resilient modulus of untreated aggregate bases is stress sensitive, the general relationship being  $M_R = K\theta^n$

where:  $M_R$  = Resilient modulus (psi)

$\theta = \sigma_1 + 2\sigma_3$  where  $\sigma_1$  is the total axial (vertical) stress and  $\sigma_3$  is the confining pressure

K & n = constants depending on material properties and test conditions.

The above references indicate most  $M_R$  values can be bounded by  $M_R = 5,000 \times \theta^{0.6}$  and  $M_R = 2,000 \times \theta^{0.6}$ . However, data from the San Diego Test Road indicate that the higher limit was exceeded by some subbase aggregate incorporated into that test road.

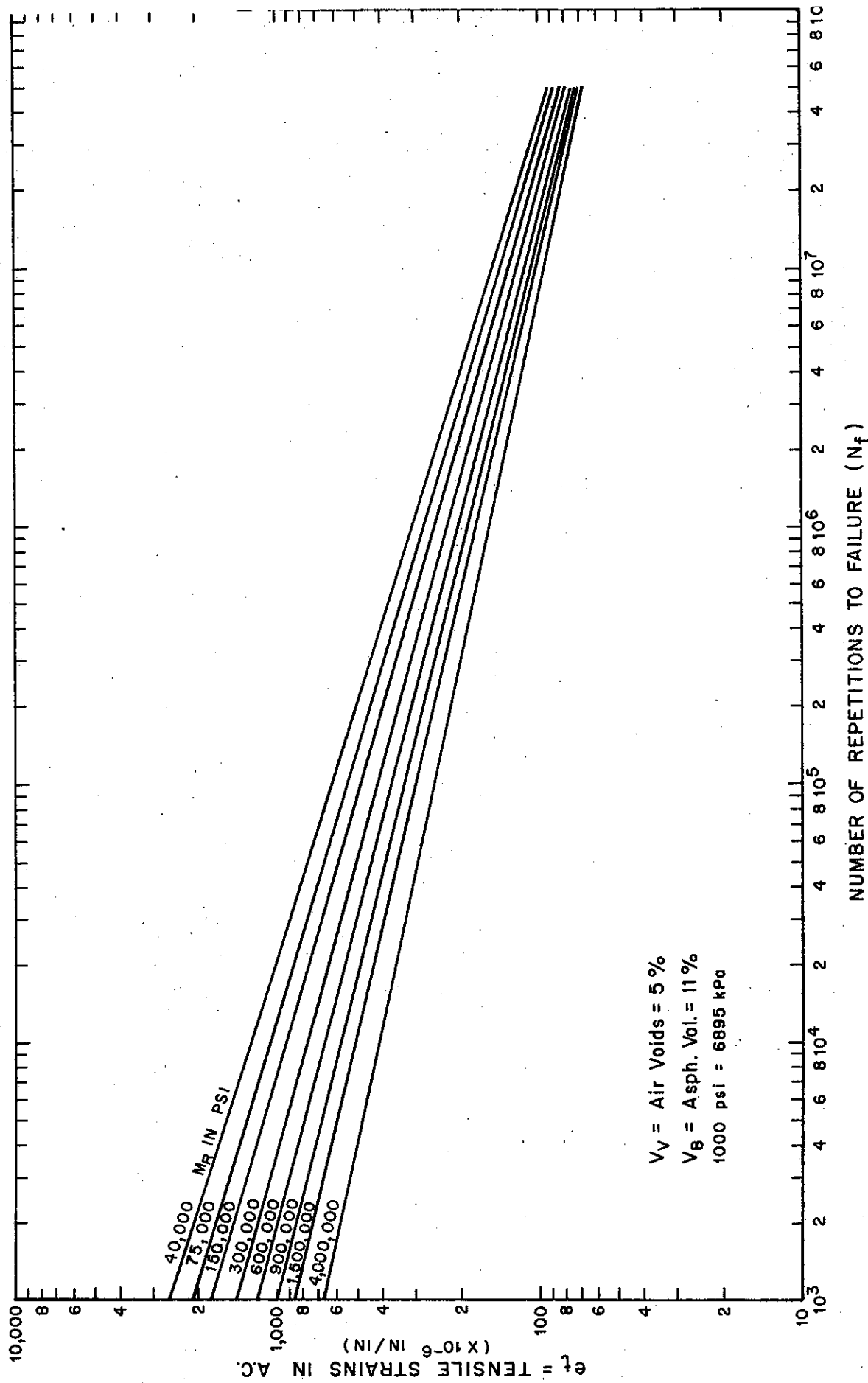


CORRELATION OF SUBGRADE MODULUS,  $E_s$ , AND R-VALUE

FIGURE 3

In the case of bases and subbases for the structural sections analyzed in this report, a K-value of 3,000 was assumed for volcanic cinder subbases and relatively thin bases (0.25 ft (76.2 mm)) over subgrade while 4,000 to 5,000 was generally assumed for the other subbases and bases in accordance with the results of the San Diego Test Road. However, the determination of the "actual" resilient modulus of the base is an iterative process so a dual wheel having a total load of 9,000 lbs (4081 kg) and tire pressure of 70 psi (483 kPa) was assumed to represent an equivalent 18,000 lb (8162 kg) axle load and each of the 5 layers (infinite depth of subgrade plus the structural section divided into 4 layers) was described by an initial elastic modulus and a Poisson's ratio. For the linear elastic AC layer,  $K = E$  and  $n = 0$ , while the values of K and n for the granular materials were assumed as previously described. The initial elastic moduli of the granular base layer was a rough approximation. In the computation, the average moduli of the granular bases was then calculated and compared to the moduli originally assumed. If these values differed by more than 2.5% a new modulus (the average of "assumed" and "calculated" values) was determined and a new set of stresses and deformations computed. This iteration was continued until the difference between modulus assumed and modulus required was less than 2.5% or until 6 "iterations" were completed. Then the calculations to obtain stresses, strains and deflections were completed.

Santucci(2) has developed a fatigue criteria chart for AC (Figure 4) which relates tensile strain at the bottom of the AC layer to number of repetitions to failure based on the stiffness (modulus of resilience) of the AC. These curves are based on laboratory tests with an allowance for crack propagation. W. van Dijk(6) determined from laboratory wheel tracking tests that the number of cycles required in the laboratory to develop cracks should be multiplied by 3 to allow for field crack propagation to the pavement



FATIGUE CRITERIA FOR ASPHALT & EMULSIFIED ASPHALT MIXES

FIGURE 4

surface for California mixes. This factor of 3 was applied by Santucci to Chevron laboratory data to obtain this chart (Figure 4). J. F. Shook found that Santucci's relationship agreed reasonably well with the performance of pavements in the San Diego Test Road(13). These pavements generally had an AC pavement thick enough so that the controlled stress flexural loading relationship (as was used for Santucci's fatigue criteria chart) was considered applicable.

However, among the pavements analyzed in this study, there were some cases where thinner AC and/or higher pavement temperatures could cause the flexure of the pavement to be more characteristic of a controlled strain mode. It is generally agreed that the fatigue life of AC is considerably longer when flexural repetition is characterized by a controlled strain mode than by a controlled stress mode. In laboratory tests by Monismith(12), for a specific initial bending strain, the difference in fatigue life can be characterized by a factor higher than 10. For the controlled strain flexure tests, the repetitions were counted up to a 50% reduction in stiffness, which was considered to be failure.

Monismith, in his study of fatigue modes(8), concluded that the controlled stress mode of loading is generally characteristic for thicker (6 inch (152 mm)) layers of AC pavement while the controlled strain mode of loading generally is predominant for thin (2 inch (51 mm)) layers of AC. Consequently, he proposed a method of characterization of type of loading called the "mode factor" (MF) where  $MF = \frac{/A/ - /B/}{/A/ + /B/}$

and: A = % change in stress due to a stiffness decrease of C  
 B = % change in strain due to stiffness decrease of C  
 C = % stiffness decrease; for purpose of this study = 40%

For controlled stress, the mode factor would be -1 and for controlled strain +1. In this study, the controlled stress mode appeared to be effective in determining pavement life for mode factors less

than -0.10. For MF values greater than -0.10 it was found that controlled strain became sufficiently influential to substantially increase the lifetime as the MF increased. A conservative relationship was developed for this comparison between mode factor and "shift factor" and is shown in Figure 5. This relationship was used to determine a prudent shift factor for the allowable repetitions determined by Santucci's fatigue relationship (Figure 4) when average mode factors greater than -0.10 occurred, meaning that the controlled strain mode began to take effect.

The vertical subgrade strains have also been related to pavement life in a study sponsored by Shell(14,15) as shown in Figure 6. Thus, the vertical subgrade strains were also calculated and used to determine allowable repetitions of vertical subgrade strain to pavement failure.

To evaluate this structural section design procedure, twenty-three existing structural sections were re-designed for the traffic they had been subjected to prior to the need for major rehabilitation and this theoretical structural section then compared with the existing structural section and with structural sections designed per current Caltrans' and AASHTO 1972 Interim Guide design procedures. Based on the four AC moduli of resilience determined for each location, the PSAD-2 method was used to determine the tensile strain at the bottom of the AC layer and the vertical strain at the top of the subgrade under a 9,000 lb (4081 kg) dual wheel loading. For each of the four moduli of resilience, the tensile AC strain was compared to Santucci's fatigue criteria chart (Figure 4) to determine allowable repetitions. The predicted lifetime of the pavement in EAL's (equivalent 18,000 lb axle loads) using 4 different moduli of resilience is  $4 / (\frac{1}{A} + \frac{1}{B} + \frac{1}{C} + \frac{1}{D})$  where A, B, C, and D are the repetitions to failure from Santucci's chart (Figure 4).

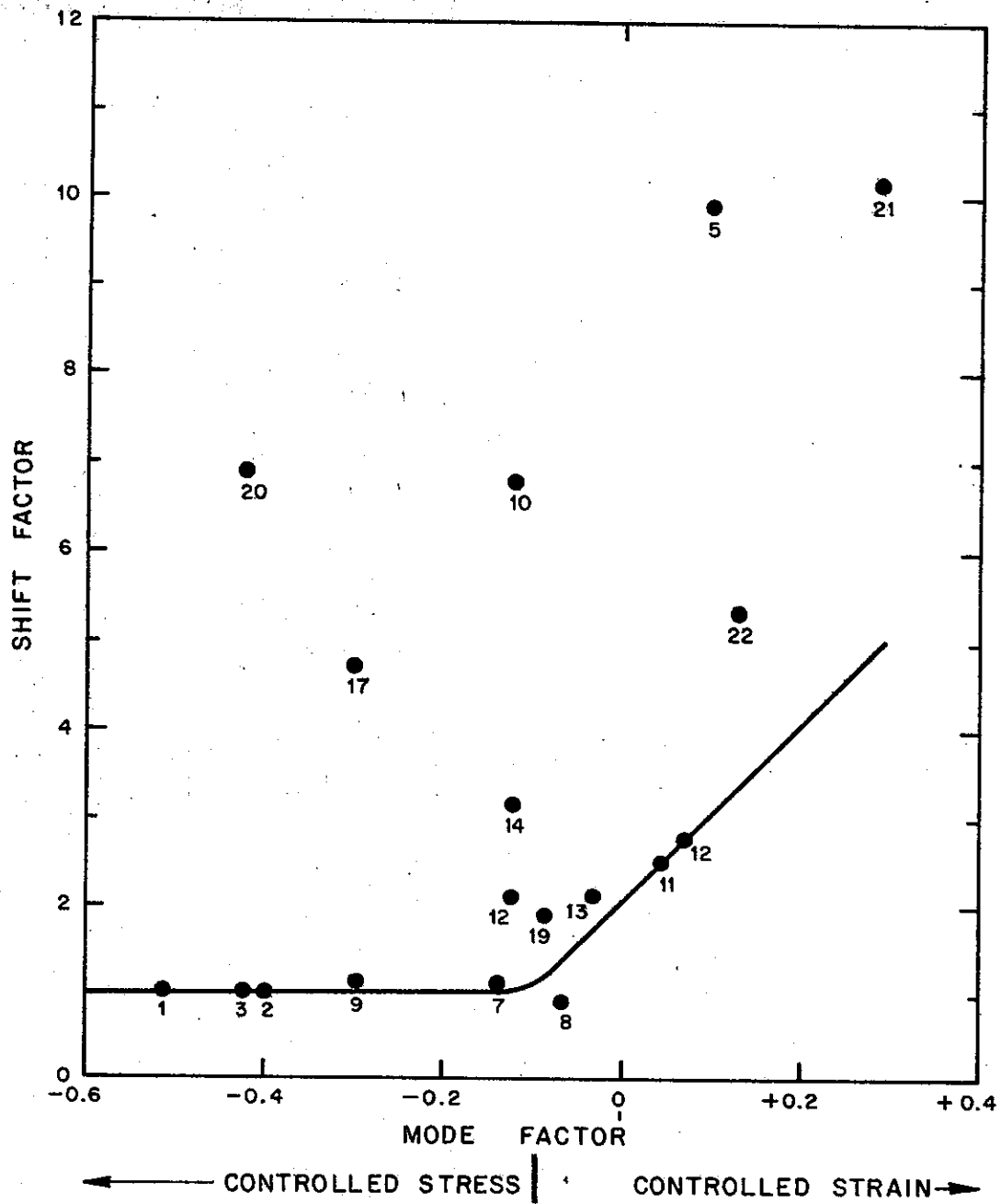
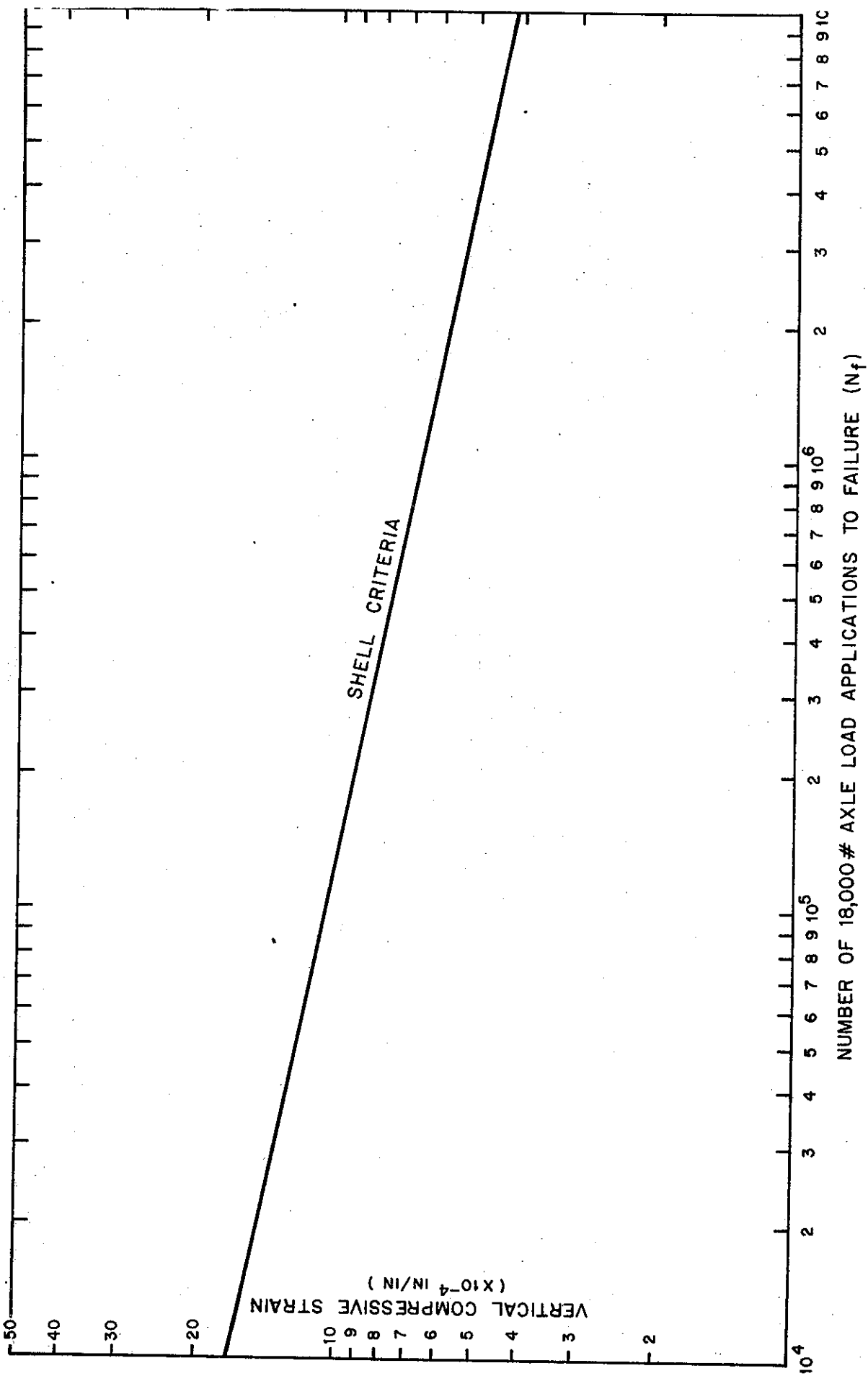


FIGURE 5



RELATIONSHIP BETWEEN SUBGRADE STRAIN AND APPLICATIONS TO FAILURE

FIGURE 6

Santucci has also developed a correction factor for air voids and asphalt volumes different from those assumed(2) based on data presented by Pell and Cooper(16) and Epps(17). The adjusted fatigue life was calculated as follows:

$$N = N_f \times 10^{[4.84 (\frac{V_B}{V_v + V_B} - 0.69)]}$$

where N = adjusted fatigue life,  $N_f$  = fatigue life (in EAL's) from Figure 4,  $V_v$  = air void volume, and  $V_B$  = asphalt residue volume. This correction can be rather severe. For example, a 2% increase in  $V_v$  can decrease  $N_f$  by 59%. Shook(13) reported that this correction would have indicated a fatigue life somewhat in excess of those observed in the San Diego Test Road for pavements with void ratios greater than 5%. However, it is probable that higher void contents have a more serious effect on performance of pavements in coastal regions of high humidity such as San Diego or in colder climates than in the dry desert regions. Research by Schmidt (44, 45) indicated that the moisture content of AC induced by 95% relative humidity can decrease its modulus of resilience equivalent to the effect of a 40°F rise in temperature, and he suggested that this in combination with the condensed water can cause acceleration of the structural damage to the pavement. Based on his research, Schmidt recommends: 1) an increase in asphalt content and decrease in void ratio, 2) cement or lime treatment of the aggregate and/or 3) emulsion treated mixes with reasonable void ratios to mitigate the effects of moisture and/or freezing and thawing.

Table 1 shows a comparison between the actual structural sections constructed for several California pavements, the theoretical structural sections required by the elastic layer method described above (using Santucci's relationship for tensile strain and the Shell criteria for vertical subgrade strain), and the structural

Table 1

Site Designation Location	Design R-Value Exud. Press. in psi	Est. Traffic to First Overlay (TI)	Load Duration K base(psi) K subbase(psi) (assumed)	Structural Section		AASHTO Interim Guide (1972)
				As Built (1978)	PSAD-2 (1978)	
1 Bakersfield Ker-Old 99 Union Ave.	32/300	9.0	0.015 sec	0.60'AC	0.60'AC 0.87'AC	0.50' AC
2 Selma Fre-Old 99 Now a City St.	70/300	11.7	0.015 sec	0.60'AC	0.66'AC 0.62'AC	0.64' AC
3 Madera Mad-99-8.0/9.0	60/300	12.2	0.015 sec	0.60' to 0.75'AC	0.74'AC 0.83'AC	0.30' AC
4 Escondido SD-78- 5.5/15.2	16/300	11.5	0.015 sec 5,000 4,500	0.28'AC 0.50'AB 1.30'ASB	0.63'AC 0.70'AC ----- -----	0.31' AC ----- -----
5 Long Beach LA-91-2.5/15	7/300	9.5	0.015 sec 5,000 4,500	0.25'AC 0.67'AB 1.08'ASB	0.50'AC 0.50'AC ----- -----	0.38' AC ----- -----
6 Castaic LA-5-61/71.7	37/300	11.8	0.015 sec	0.33'AC	0.67'AC 1.08'AC	0.72' AC
7 Foothill Blvd. LA-210- 34.7/35.7	65/300	9.5	0.015 sec	0.33'AC	0.37'AC 0.54'AC	0.47' AC
8 Ventura Ven-23- 10.9/12.9	25/300	7.8	0.015 sec 5,000	0.25'AC 0.67'AB	0.25'AC 0.55'AC -----	0.22' AC -----

Table 1 (Continued)

Site Designation Location	Design R-Value Exud. Press. in psi	Est. Traffic to First Overlay (TI)	Load Duration K base(psi) K subbase(psi) (assumed)	Structural Section		AASHTO Interim Guide (1972)
				As Built	PSAD-2 Calif Des. (1978)	
9 Cuesta Grade SLO-101-33/35	12/300	10.0	0.050 sec 5,000 4,500	0.60'AC 0.60'AB 1.60'ASB	0.60'AC 0.37'AC	0.13' AC
10 Gonzales Mon-101- 67.7/73.8	19/300	11.0	0.015 sec 5,000 4,500	0.33'AC 0.50'AB 1.25'ASB	0.52'AC 0.85'AC	0.25' AC
11 Cinco Ker-14-35/39.5	60/300	8.8	0.030 sec 3,000	0.25'AC 0.25'AB	0.25'AC 0.46'AC	0.36' AC
12 Independence Iny-395-65/73	60/300	8.5	0.015 sec	0.25'AC	0.32'AC 0.52'AC	0.40' AC
13 Big Pine Iny-395-var.	30/300	8.6	0.015 sec 5,000 3,000	0.25'AC 0.50'AB 1.50'ASB	0.27'AC 0.20'AC	0.25' AC
14 Manzanar Iny-395-var.	60/300	9.0	0.015 sec	0.25'AC	0.38'AC 0.56'AC	0.45' AC
15 Bridgeport Mno-395-var	21/300	6.8	0.050 sec 3,000	0.25'AC 0.25'AB	0.25'AC 0.60'AC	0.30' AC
16 Clements SJ-12-23/27.5	36/300	9.3	0.015 sec 5,000	0.25'AC 0.50'AB	0.25'AC 0.67'AC	0.35' AC
17 Marysville Yub-70-0.1/2.0	15/300	9.5	0.015 sec 5,000 4,500	0.33'AC 0.67'AB 1.00'ASB	0.52'AC 0.40'AC	0.17' AC
18 Old Rte 245 Sut-99-8/14	5/300	9.0	0.015 sec 5,000	0.25'AC 0.67'AB	0.68'AC 0.85'AC	0.68' AC

Table 1 (Continued)

Site Designation Location	Design R-Value Exud. Press. in psi	Est. Traffic to First Overlay (TI)	Load Duration		Structural Section As Built	Structural Section PSAD-2 Calif Des. (1978)	AASHTO Interim Guide (1972)
			K base(psi)	K subbase(psi) (assumed)			
19 Clear Lake Lak-53-1/7	10/300	8.0	0.015 sec 5,000 4,500		0.25'AC 0.50'AB 1.00'ASB	0.38'AC 0.38'AC	0.13' AC
20 Indio Riv-10-62/86	26/400	10.5	0.015 sec. 5,000 5,000		0.33'AC 0.67'AB 1.00'ASB	0.85'AC 0.42'AC	-
21 Indio Riv-10-62/86	19/300	10.5	0.015 sec 5,000 5,000		0.33'AC 0.67'AB 1.00'ASB	0.85'AC 0.55'AC	0.11' AC
22 Brawley Imp-115- 21.5/35	23/400	10.0	0.015 sec 5,000 5,000		0.40'AC 0.83'AB 1.00'ASB	0.60'AC 0.37'AC	-
23 Brawley Imp-115- 21.5/35	16/300	10.0	0.015 sec 5,000 5,000		0.40'AC 0.83'AB 1.00'ASB	0.60'AC 0.38'AC	0.63' AC

Note: R-value adjusted as indicated in Ref. 9 where required.

100 psi = 689 kPa  
1.0 ft = 305 mm

sections required per the 1972 AASHO Interim Guide and the present California design method. The estimated traffic index prior to construction of the first overlay was used for these calculations.

The elastic layer method provided structural sections more similar to those actually constructed than did the current Caltrans' procedure in most cases. The present design method using R-values for 300 psi (2068 kPa) exudation pressure, however, appears to provide answers that agree more closely with the performance of the four pavements in the hottest area (see Sites 20-23, Table 1) while the elastic layer design, as presented in this study, requires thicknesses of AC considerably greater than those needed. However, some research(11,18,19) indicates that the elastic layered theory is less appropriate at higher temperatures and, therefore, this fact must be taken into consideration when designing pavement in areas such as Brawley and Indio which have average annual temperatures of approximately 72°F (22°C). It is possible that Santucci's tensile strain-fatigue relationship is too conservative for warm climates where viscoelastic flexure appears to occur. It therefore appears that additional research is needed to determine the necessary modifications for the hotter regions of California.

In regions where annual freezing and thawing combined with moderate rainfall occurs, it is necessary to especially consider the effects of these changes on the subgrade if the elastic layer method is to be used properly in the design. Studies by Bergan and Monismith(8) made on pavements in Canada revealed that for these colder climates, a load duration of 0.05 seconds to determine stiffness of AC for the elastic layer analysis gave results that were indicative of the performance of these pavements so, as stated previously, this load duration was used for analysis of a pavement in Bridgeport in Mono County (Table 1 - Site 15). The tensile strain versus allowable

repetitions relationship used in the Canadian study was based on repetitive flexure to initiation of cracking and did not consider crack propagation. This type of test is probably more characteristic of pavement in very cold climates. The extremely moist subgrade during the thawing period, the cold temperature cracking of the AC, and frost heaving could all combine to significantly influence crack propagation rate. The average annual temperature in this area is about 43°F (6°C) and the average annual rainfall is 9.5-inches (228 mm), both of which are considerably less than those for major portions of California. The results of the rational design agreed well with performance.

Pavements near Bieber and Lassen National Park in Lassen County, where average temperatures are also about 43°F (6°C) and the average annual precipitation is 23 inches (585 mm), were also analyzed. The corresponding figures for Regina, Canada, where the aforementioned study(8) was made, are 36°F (2°C) and 17 inches (432 mm). Since the rainfall is somewhat greater in Lassen County than in Saskatchewan, Canada, the effect of frost heave and freezing and thawing due to moisture in the subgrade in Lassen could be more severe than that in Saskatchewan. For this reason, Santucci's curve (Figure 4) was modified for this condition by dividing the allowable repetitions by 3 to represent the initiation of cracking. Table 2 shows a comparison of the actual traffic life before overlay and the theoretical fatigue life considering constant subgrade  $M_R$  for R-value at 300 psi (2068 kPa) exudation pressure. These results indicate that the theoretical EAL's to failure would be considerably greater than the actual traffic for the two cases where freezing and thawing occurs if the allowance (factor of 3) for crack propagation assumed by Santucci is used. Table 2 also shows the structural sections as built in Lassen County and the structural sections as they would have been designed (for the traffic carried) using the present California design procedure. These structural section designs were very similar. It was then assumed that the modulus of resilience of the subgrade in Lassen County would vary, due to

Table 2

## Performance of AC Pavement in Cold Freeze-Thaw Areas

Location	Strength	Estimated Traffic (TI)	Average Annual Temp.	Average Annual Precip.	Structural Section		Ratio Actual/EAI Theor.
					As Built	Current Caltrans	
Bieber Lassen 299	R-value = 31 M <sub>R</sub> = 12.0 ksi	7.0	46.5°F	23"	0.25'AC 0.50'AB 0.75'ASB	0.25'AC 0.50'AB 0.40'ASB	0.58
Lassen Park Lassen 44	R-value = 24 M <sub>R</sub> = 8.9 ksi	6.6	43.5°F	23"	0.20'AC 1.00'AB	0.24'AC 1.00'AB	0.91

Notes: 1 ksi = 6.89 MPa  
 1" = 25.4 mm  
 46.5°F = 8°C  
 43.5°F = 6°C

the effects of freezing and thawing, in a way similar to that at the Canadian location. Benkelman beam deflection profiles had been obtained for measurements taken during the time the Saskatchewan road was in operation and then used to determine subgrade modulus by analytical techniques (CHEV 5L). Results of these determinations are given in Table 3. This data can be considered only as a rough approximation because of the assumptions that were then made such as an AC  $M_R$  of 300,000 psi (2068 MPa). Similar variation in subgrade  $M_R$  was assumed to analyze the pavements in Lassen County by the elastic layer method assuming the highest  $M_R$  value just prior to freezing corresponded to the theoretical  $M_R$  for R-value at 300 psi (2068 kPa) exudation pressure. The ratio of the actual versus the theoretical EAL's was 1.12 for Route 299 and 3.94 for Route 44. Thus, this assumption of variable subgrade modulus also resulted in somewhat inconclusive results regarding the applicability of the theoretical procedure to cold freeze/thaw regions.

Per Table 1, the present Caltrans' design procedure would have provided an inadequate AC thickness for four of the twenty-three structural sections (Sites 9, 13, 22, and 23). In all cases, the structural section dictated by the theoretical approach under study here would have apparently been adequate. In addition, the average AC thickness per the theoretical approach for these twenty-three structural sections was 144% of that actually constructed, whereas the current Caltrans' design procedure would dictate AC thicknesses for the nineteen "adequate" structural sections that would be 187% of that actually constructed. Interestingly, use of the AASHO procedure resulted in average structural section thicknesses 106% of that actually constructed. However, the standard deviations of the differences between the as-built thicknesses and the thicknesses obtained using the three design procedures were 0.157' for the PSAD-2 design, 0.227' for the 1978 California procedure, and 0.235' for the AASHO procedure.

Table 3  
Determination of the Subgrade Modulus  
Regina to Lumsden

<u>1969 Month</u>	<u>Deflection inches <math>\bar{X} + 2\sigma</math></u>	<u>Subgrade Modulus psi.</u>
April	0.035	5800
May	0.054	2850
June	0.0525	3200
July	0.0435	4300
August	0.0365	5500
September	0.030	7100
October	0.027	8400

Resilient modulus of subgrade determined by using the CHEV5L Program with deflection controls.

Other parameters: - Stiffness of asphalt concrete - 300,000 psi.  
Base and subbase parameters as determined in Chapter 6, Ref. 8.

thus indicating that the PSAD-2 procedure provided better estimates of the AC thicknesses actually needed than did either method. The "excess" thickness per the theoretical approach may have been less, at least in some cases, if the design had been adjusted for average in-service void contents. Because of the lack of field data for these pavements, no adjustment of the assumed 5% AC void content could be made. The data indicates that, in most instances, the design AC thickness using either approach would be excessive. There was occasional good agreement between the results of the PSAD-2 and Caltrans' methods (see Sites 2, 5, 13, and 19) and several examples of very poor agreement (see Sites 6, 7, 9, 15, 16, 20, and 22).

In two of the design comparisons (Sites 9 and 13), the California flexible pavement design method did not provide an adequate structural section and for the other two (Nos. 22 and 23), the design was marginal. Because of the steep slopes at Cuesta Grade (which, according to Figures 1 and 2, correspond to 0.05 sec load duration), the required thickness of AC determined by PSAD-2 was equal to that provided for the traffic it carried, while the current Caltrans' design method would have provided a very inadequate structural section for the actual traffic carried. Also, the comparison of the two design methods indicates that the gravel equivalent method is not always adequate to characterize thick untreated base layers.

One of the significant findings of the San Diego Test Road was that the untreated bases (for a given thickness of AC surfacing) failed in inverse order of their thicknesses. Performance data on the Brampton Test Road(20) and elsewhere(21) in Canada verified that increases in untreated base thicknesses often resulted in poorer performance. Finn has concluded in reviewing various rational design methods(22), that the PSAD-2 method is one of the best analytical pavement design tools in current use. Using the PSAD-2 method of analysis to characterize the bases

of the San Diego Test Road, Hicks and Finn(23) confirmed that maximum asphalt tensile strain could increase with this increased base thickness.

In a comparison of the AASHO 1972 Interim Guide flexible pavement design method with performance at the San Diego Test Road, it was determined that the AASHO design would have provided a conservative structural section for about 80% of the pavements tested. However, the maximum equivalent TI for the San Diego tests was 7.5 while the TI range for the pavements analyzed in this study was 7.8 to 12.2. A comparison of the required structural sections as determined by the AASHO Interim Guide (1972) with the sections as placed indicates that it would have provided conservative answers for only 57% of the pavements described in Table 1. For five of the seven "full depth" AC test sections, the AASHO design method gave conservative designs. However, only one of the other eight structural sections that would have required an AC thickness less than that furnished had less than 1.5 ft of total base. This is an indication that the layer coefficients ("a") for the AC appear to be reasonable for this design method when used for AC constructed per current California standards, but the respective layer coefficients appear too liberal for the aggregate bases (AB) and aggregate subbases (ASB). The layer coefficient of AB is about 1/3 that of AC for the AASHO method, while for the California method, the ratio of the gravel equivalent of AB to AC is about 0.55 for a TI of 8. Therefore, the AB to AC equivalency ratio is 67% higher for California design than for the AASHO method, making it even more liberal for the layer equivalency for aggregate base when compared to that of AC. This is another indication that the California gravel equivalent method for structural section design, though conservative, does have limitations.

Generally, the current Caltrans' procedure provided the thickest AC in those instances where the agreement between design and as-built structural sections was poor. The theoretical design method generally appears to give a somewhat more reasonable structural section design for California pavements than the present method, except when very hot ( 70°F) or very cold ( 50°F) climates are encountered. These climatic extremes are also not taken into account by the current Caltrans' procedure.

The complexity and, presumably, the cost of the theoretical flexible pavement design procedure would be considerably greater than that of the current procedure, when taking materials testing costs into account. Also, the comparison presented herein was based upon data obtained using numerous estimates and assumptions of as-built material's characteristics that were not available. In addition, it appears as though more research needs to be conducted to determine the effects on pavement performance of void ratio, freezing and thawing, viscoelastic flexure, age hardening and excessive moisture in AC. This information is needed to make the elastic layer design method more responsive to the variations in asphalt, construction methods and climate in California. For these reasons, plus the fact that both the current and the theoretical procedures appear to provide somewhat excessive thicknesses, adoption of the theoretical procedure in lieu of the current procedure for routine design does not appear to be warranted. The theoretical procedure does, however, provide a method to use when analyzing the ability of an existing structural section to support overloads and it could be used for these problems.

#### B. Semi-rigid Pavement Design

The most recent research on cement treated base (CTB) in the U.S., Great Britain, and South Africa(24,25,26,27) indicates that in

pavement consisting of AC over CTB, generally the strains in both the AC and subgrade or subbase are less than the safe working values, and, therefore, the stresses and strains in the cement treated layer are the limiting values. An investigation of the performance of cement treated bases in California(28) indicates that pavements that had CTB with higher compressive strength tended to have less block cracking, thus supporting this contention.

In many of the design procedures involving CTB or lean concrete base (LCB) pavements, it is assumed that there is load transfer at the shrinkage cracks in the CTB or LCB. L. D. Childs of the PCA(29) indicates that there definitely is load transfer at the transverse joints of non-faulted rigid pavements even when no dowels are used. Also, the PCA design method for airport pavements relies heavily on this load transfer at the joints. Considerable work in the finite element method of design for rigid pavement has been done by Y. H. Huang and S. T. Wang of the University of Kentucky(30). Their study indicates that in pavements where the outer face of the tire is near the edge, the most critical stress occurs at a considerable distance from the joint if load transfer is provided at the joint. Studies at the Universities of California(31) and Illinois(32) verify this. The studies at the Universities of Kentucky(30) and Illinois(32) both suggest edge stress be used in the design of rigid pavements. The results of the Illinois study indicated that although only 5 to 10% of the wheels pass within 6-inches of the slab edge, they account for the majority of the fatigue damage.

The characteristics of a pavement having a thick layer of CTB or LCB under an AC surfacing would seem to be more related to "rigid" pavement than to "flexible" pavement. For this reason, the classical theory of thin rigid plates on "Winkler" foundations as employed by Westergaard(33) appeared to be justified for these

pavements. This assumption was used by Huang and Wang(30) in developing the finite element solution and the results agreed with results obtained using the Westergaard method and experimental measurements from the AASHO road test. Repeated short-duration load tests on cement stabilized soil slabs by Mitchell, et al(25) and Fossberg(31) indicated that finite element or elastic layer analysis could be used to accurately calculate interior load stresses and strains based on material properties determined from laboratory repeated load tests on undisturbed specimens taken from the pavements. Mitchell's studies also show that tensile stresses determined by the Westergaard theory check closely with those determined by the elastic layer analysis. These studies also indicate that loads placed 2 feet (635 mm) or less from the pavement edge will result in larger stresses than those occurring due to interior wheel loads.

An investigation of cement treated base performance in California(28) indicated that block cracking in the pavement was significantly reduced by extending the CTB at least 1 foot (293 mm) into the shoulder. Cracking was most prevalent near the outer wheel track. Diagrams of cracks in the thicker concrete pavements in the AASHO Road Test also seem to indicate cracking began at the edge of the pavement. L. D. Childs'(29) results also support extending "cement treated soil" base 1 foot (293 mm) beyond the edge of concrete pavement. These facts would seem to justify the assumption that the critical flexural stress is dependent on the proximity of the outer wheel to the edge of the structural section.

The results of compressive strength tests performed on a construction project near Compton(34 - see Table 4) were used to obtain comparative strength and stiffness data for design comparisons between CTB and LCB based on the Westergaard method of design. To provide a safety factor for the design, the 20th percentile 90 day compressive strength (the stress which 80% of the 90 day cores could sustain without

Table 4

COMPRESSIVE STRENGTHS (psi)  
(100 psi = 689 kPa)

Field Fabricated Samples		Cores from Pavement	
7 day	28 day	28 day	90 day
<u>6" x 6" Samples</u>		<u>LCB 5" Cores</u>	
149	245	610	590
157	245	642	610
161	252	663	645
384	549	668	715
385	570	683	750
388	577	694	775
399	577	705	865
405	605	711	900
412	629	716	900
455	654	764	935
465	685	886	945
469	692	<u>1061</u>	<u>990</u>
Avg. 352	523	734	802
<u>4" x 4" Samples</u>		<u>CTB 5" Cores</u>	
732	931	504	465
740	995	509	475
909	1011	509	555
916	1019	512	555
920	1074	530	560
956	1074	570	615
963	1082	594	615
990	1106	647	685
1000	1106	647	705
1032	1170	700	790
1052	1210	748	815
1059	1281	<u>870</u>	<u>985</u>
Avg. 939	1088	612	652

Note: 1" = 25.4 mm

failure) was used. This was determined to be 550 psi (3792 kPa) for the CTB and 660 psi (4551 kPa) for the LCB.

In determining the stiffness of the LCB or CTB, the empirical formula  $E = 33(W)^{1.5} \sqrt{f_c'}$  was used, W being the density of the base and  $f_c'$  its unconfined compressive strength. J. K. Mitchell, who has done considerable testing of CTB, states that the flexural strength of CTB is between 25% and 35% of the compressive strength and presents a formula for flexural strength which is  $1/2(f_c')^{0.88}$ . Using these relationships, the stiffnesses and flexural strengths of the LCB and CTB used in this study were calculated as follows:

$$E_{LCB} = 33 (W)^{1.5} \sqrt{f_c'} = 33(146)^{1.5} 660 = 1,500,000 \text{ psi (10,342 MPa)}$$

$$f_{b'}_{LCB} = 1/2 (f_c')^{0.88} = 1/2 (660)^{0.88} = 152 \text{ psi (1048 kPa)}$$

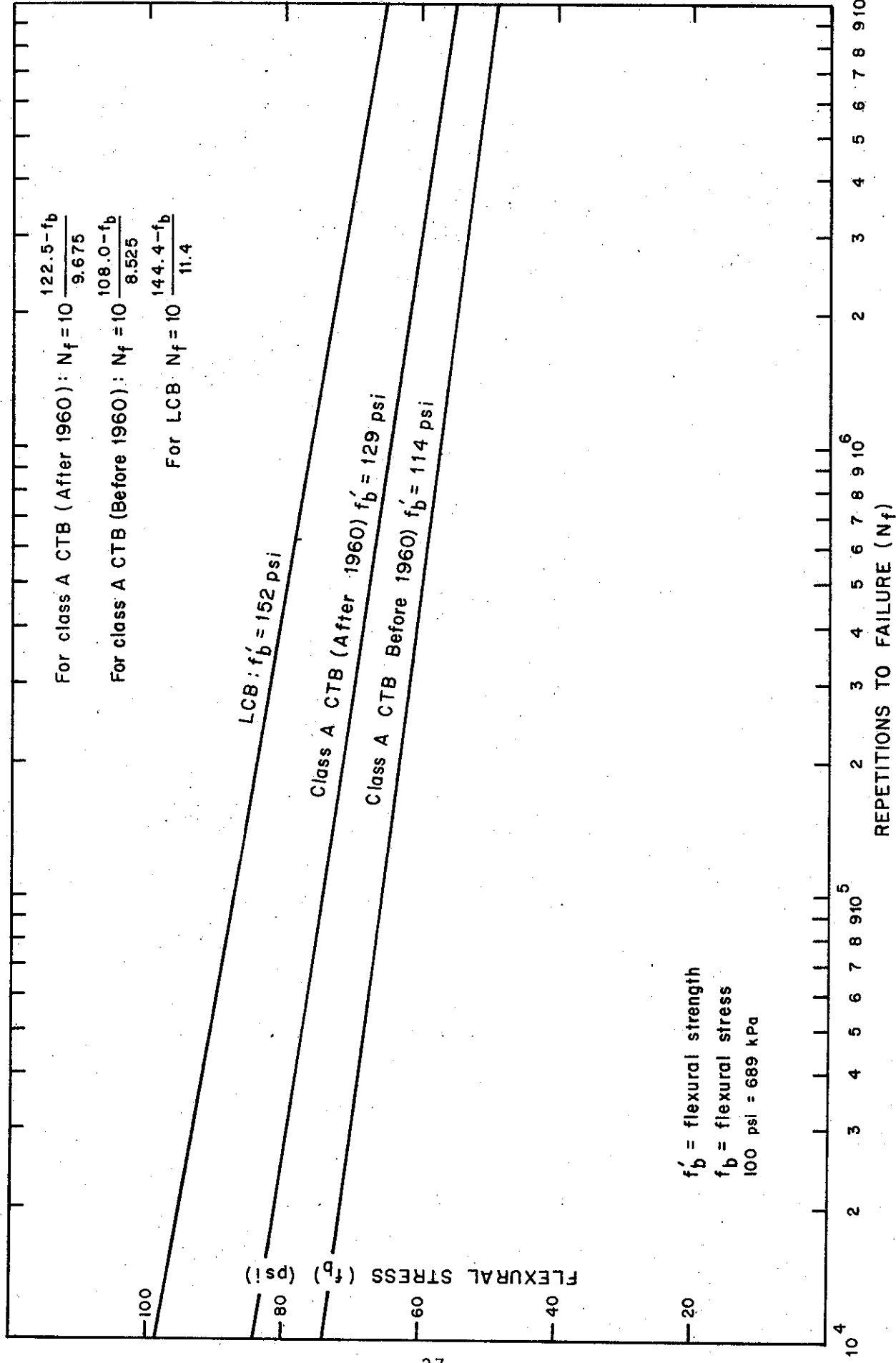
$$E_{CTB} = 33 (W)^{1.5} \sqrt{f_c'} = 33(138)^{1.5} 550 = 1,250,000 \text{ psi (8618 MPa)}$$

$$f_{b'}_{CTB} = 1/2 (f_c')^{0.88} = 129 \text{ psi (889 kPa)}$$

The stiffness of the AC surfacing was determined using the same program used for the flexible pavement analysis described previously and the AC and CTB then considered a composite section. Four different AC stiffnesses were used for each pavement corresponding to seasonal variations. Fossberg(31) found good agreement between measured and calculated stresses and strains when assuming composite action and using finite element and elastic layer analysis. Otte'(27) also suggests that the assumption of composite action is appropriate.

Allowable repetitions for various CTB stress ratios (ratio of flexural stress to flexural strength) presented by Pretorius, Mitchell, and Otte'(24,25,27) agree closely with the conservative relationship presented by PCA for concrete. The stress ratio suggested for 100 repetitions by PCA was 80% whereas for  $10^6$

repetitions it was 50% based on the findings of these three authors. The plot of stress versus repetitions to failure for CTB and LCB derived for this study is shown in Figure 7. The stresses in the LCB or CTB were obtained from the influence chart for moments at the edge of a rigid pavement (published by PCA) based on the Westergaard rigid plate theory.



CTB - LCB DESIGN GUIDE  
FIGURE 7

Pavement polishing as revealed by studies in California show the outer wheel path to be between 2 and 3 feet (0.61 to 0.91 m) from the edge of the travelled way. However, in order to provide a sufficiently conservative model for design, it was assumed for this study that the edge of the passing dual wheel tire was 6 inches (152 mm) inside the edge of the travelled way, or 18 inches (457 mm) from the edge of the CTB layer since it extends 1 foot (304 mm) outside the travelled way. It was assumed that this load location would provide a factor of safety sufficient to compensate for temperature stresses even though Mitchell(25) states that generally temperature stresses do not affect the fatigue life appreciably.

Basically, the Westergaard edge loading design procedure is similar to the PCA concrete pavement design method now in use in California. The CTB tensile stresses are determined for each axle load considered and the percent fatigue resistance attributed to each axle load determined and summed up according to Miner's rule. In order to check the proposed design method, theoretical designs were developed and compared using estimated traffic intensity for some of the AC/Class A CTB pavements included in the 1968 California Division of Highways' study(28) and some other similar pavements with more recent traffic patterns and longer pavement lives. The results of this effort are shown in Table 5. This table also includes designs prepared using the present Caltrans' design method. In order to get a true comparison, it was necessary to adjust all except one of the original R-values (which were determined on the basis of 400 psi (2757 kPa) exudation pressure) to R-values at 300 psi (2068 kPa) exudation pressure per Reference 9 when developing designs per present Caltrans' criterion. In addition, an unconfined compressive strength of 750 psi (5169 kPa) was assumed for the CTB and the traffic pattern used in the Westergaard approach for the pavements overlaid before 1968 (Table 5) was assumed to be similar to that taken from Caltrans' loadometer data (All Main Rural) for 1962. For the more recently overlaid pavements, more recent traffic patterns were used.

Table 5

Site Designation	Job Location County-Rts. Contract No.	Subgrade & Base R @ 300 psi Ex. k under CTB	Est. Traffic to Overlay TI	Structural Section	
				As Built	Westergaard Edge Loading Calif. Des. (1978)
1	Yorkville Men-128 58-1TC2	$\frac{25}{87}$	8.1	0.20' AC 0.50' CTB	0.75' CTB 0.88' CTB
2	Wilson Cr. DN-101 57-1TC4	$\frac{16}{103}$	8.9	0.20' AC 0.67' CTB 0.50' ASB	0.67' CTB 0.88' CTB
3	Wilson Cr. DN-101 57-1TC4	$\frac{5}{145}$	8.9	0.20' AC 0.67' CTB 1.00' ASB	0.64' CTB 0.78' CTB
4	Eureka Hum-101 55-14TC33	$\frac{40}{110}$	9.2	0.20' AC 0.50' CTB	0.64' CTB 0.83' CTB
5	Willits Men-101 52-1TC7	$\frac{17}{133}$	8.9	0.25' AC 0.50' CTB 0.75' ASB	0.58' CTB 0.62' CTB
6	Ridgewood Men-101 54-1TC18	$\frac{18}{135}$	8.7	0.20' AC 0.50' CTB 0.75' ASB	0.63' CTB 0.70' CTB
7	Willits Men-101 57-1TC9	$\frac{30}{92}$	8.8	0.20' AC 0.67' CTB	0.73' CTB 0.94' CTB
8	Delavan Col-Gle-5 55-3TC6	$\frac{40}{110}$	8.4	0.21' AC 0.42' CTB	0.71' CTB 0.78' CTB

Table 5 (Continued)

Site Designation	Job Location County-Rts. Contract No.	Subgrade & Base R @ 300 psi Ex. k under CTB	Est. Traffic to Overlay TI	Structural Section	
				As Built	Westergaard Edge Loading Calif. Des. (1978)
9	Applegate Pla-80-24.7/27.4 55-3TC13F	22 84	8.8	0.25' AC 0.67' CTB 0.17' ASB	0.75' CTB 1.00' CTB
10	Madison Yol-505 13.3/17.4 55-3TC12	8 87	8.9	0.21' AC 0.67' CTB 0.42' ASB	0.77' CTB 1.00' CTB
11	Napa Nap-29 58-4TC3	17 106	7.9	0.25' AC 0.50' CTB 0.50' ASB	0.61' CTB 0.70' CTB
12	Encinitas SD-5 57-11VC28	25 87	7.9	0.25' AC 0.67' CTB	0.67' CTB 0.80' CTB
13	Davis Sol-Old Rte. 7 56-10CBC-1	20 80	9.3	0.27' AC 0.67' CTB	0.78' CTB 1.14' CTB
14	Mossdale SJ-5 59-10TC3	18 80	9.7	0.23' AC 0.67' CTB	0.78' CTB 1.25' CTB
15	Carpinteria Ven, SB-101 54-7VC73	20 80	8.9	0.33' AC 0.67' CTB	0.65' CTB 0.95' CTB
16	S.FK.Eel R. Hum-101-36.0/39.0 58-1TC3	17 137	10.1	0.27' AC 0.67' CTB 0.75' ASB	0.70' CTB 1.30' CTB

Table 5 (Continued)

Site Designation	Job Location County-Rts. Contract No.	Subgrade & Base R @ 300 psi Ex. k under CTB	Est. Traffic to Overlay TI	Structural Section	
				As Built	Westergaard Edge Loading Calif. Des. (1978)
17	S.FK.Eel R. Hum-101-36.0/39.0 58-1TC3	26 116	10.1	0.27' AC 0.67' CTB 0.50' ASB	0.72' CTB 1.10' CTB
18	Fernbridge Hum-101-62.0/64.0 58-1TC5	15 100	10.1	0.27' AC 0.67' CTB 0.50' ASB	0.72' CTB 1.00' CTB
19	Arnold Men-101-55.0/60.0 55-1TC21F	22 170	10.0	0.33' AC 0.67' CTB 1.00' ASB	0.70' CTB 0.60' CTB
20	Laytonville Men-101-73.0/75.0 55-1TC3	17 78	8.9	0.24' AC 0.67' CTB	0.75' CTB 1.10' CTB
21	Cottonwood Teh-5-28.0/42.0 63-2TL3C4	14 200	11.0	0.42' AC 0.67' CTB 1.40' ASB**	0.70' CTB* 0.55' CTB**
22	Marysville Yub-65, 70 57-3TC	5 145	9.9	0.27' AC 0.67' CTB 1.00' ASB	0.75' CTB 0.86' CTB
23	Roseville Pla-80 57-3TC27F	17 103	10.7	0.25' AC 0.67' CTB 0.50' ASB	0.87' CTB 1.13' CTB
24	Colfax Pla-80 57-3TC21F	13 84	10.3	0.27' AC 0.67' CTB 0.33' ASB	0.90' CTB 1.18' CTB

Table 5 (Continued)

Site Designation	Job Location County-Rts. Contract No.	Subgrade & Base R @ 300 psi Ex. k under CTB	Est. Traffic to Overlay TI	As Built	Structural Section Westergaard Edge Loading (1978)
25	Inglewood LA-42-2.0/4.4 55-7VC20	18 103	8.9	0.33' AC 0.67' CTB 0.50' ASB	0.62' CTB 0.70' CTB
26	I5-Rte 101 & 110 LA-101-0/0.7 50-17VC131	15 76	10.8	0.50' AC 0.67' CTB	0.77' CTB 1.40' CTB
27	Visalia Tul-63-6.5/8.7 47-6XC12	39 240	7.8	0.25' AC 0.50' CTB	0.52' CTB 0.59' CTB

Notes: \*Thickness required for edge loading

\*\*Thickness of ASB in truck lane varied between 1.20' minimum to 1.45' maximum at shoulder.

100 psi = 6894 kPa, 1.0' = 305 mm

The average thicknesses of CTB required per the present design method is about 3 inches (76 mm) more than that required by the Westergaard method. The CTB thickness required by the Westergaard method is an average of one inch (25 mm) more than the as-built CTB thickness. If CTB having the strength required by present specifications had been used for those pavements built before 1960, the average difference between the as-built and design (Westergaard) thicknesses would have been even greater than 1-inch (25 mm) indicating that both methods appear conservative for the traffic loading of the 1960's. However, the present design method would have provided inadequate structural sections for pavements in Tehama County and Mendocino County (Sites 21 and 19). The pavement in Tehama County, which contained the higher strength CTB, carried the heaviest traffic included in this study and is the most characteristic of more recent, heavier loading. The pavement in Mendocino County also carried considerably heavier loads in the later years of its life. These two pavements also had thick aggregate subbases, which may not be as effective structurally as indicated by the present design method.

Traffic indices (TI's) based on EALs (equivalent 18,000 pound (8162 kg) axles per the California flexible pavement design criteria) were used in Table 5 in order to relate the present design method to the Westergaard method. The present design method characterizes the EAL's for any axle load as  $(\frac{W}{T_8})^{4.2}$ , where W is the weight of the axle in question. Monismith(35) describes the repetitions to fatigue failure by laboratory test for California AC as being approximately inversely proportional to the initial bending strain cubed ( $e^3$ ) whereas Pretorius(24) determined an approximate inverse proportionality for CTB of  $e^{20}$  from flexural laboratory tests. From these relationships, it can be seen that the Westergaard design method for AC/CTB pavements should become more conservative than the present method as axle loads increase.

In order to illustrate the effect of an increase in axle loads as related to structural sections required under the present and the theoretical design method, a comparison of the two structural section design procedures is shown in Figure 8. The modulus of resilience of the AC was assumed to be 500,000 psi (3450 MPa).

Examination of this comparison indicates, as did the data in Table 5, that the present design method dictates the use of a considerably greater thickness of CTB than does the theoretical method at the more common higher TI values now being encountered - i.e., heavier traffic. Because the performance data in Table 5 indicates that the thicknesses provided by the theoretical method would be adequate, this suggests that as traffic becomes heavier, the theoretical approach will become increasingly more attractive.

The slope of the theoretical curves is similar to that obtained using the PCA design method(36), which is based on fatigue data obtained from repeated loadings applied to CTB slabs as interior loadings rather than edge loadings. The flexural strength for the CTB composed of aggregate and 7% cement in the PCA tests was similar to that assumed for the Class A CTB constructed under present specifications. A comparison of the theoretical (Westergaard) design method and the PCA design method is shown in Figure 9. It can be seen that for both design methods, the slope of the curves for given subgrade support values is approximately the same and that the theoretical approach under consideration will provide greater thickness for any given traffic density. The results on Table 5 indicate, however, that these larger thicknesses are appropriate.

As the traffic pattern becomes more dense and the loads increase, the shortening of rest periods between loads can shorten the life of concrete pavements considerably, according to laboratory flexural fatigue tests by Hilsdorf and Kesler(37). They also determined that

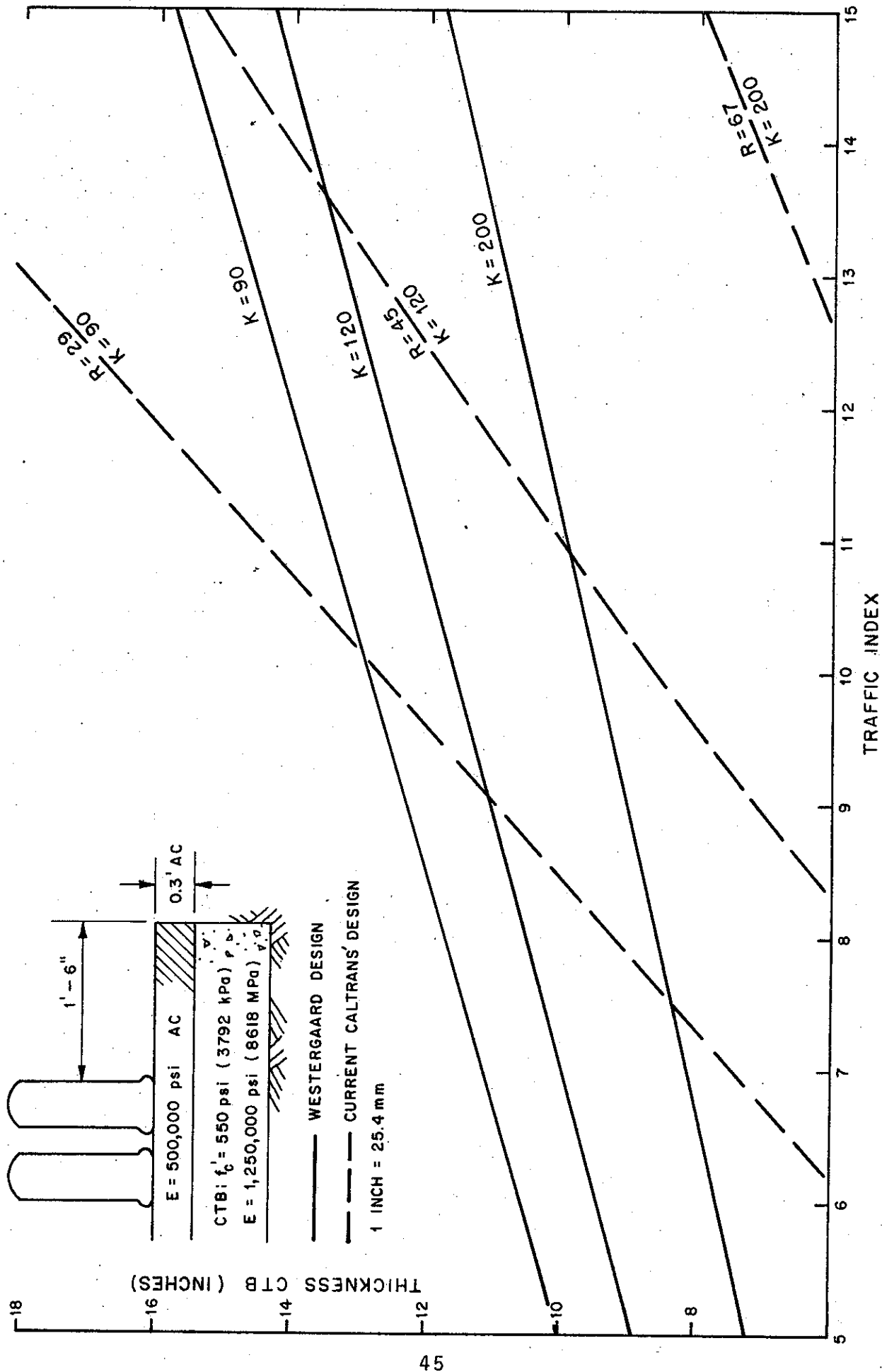
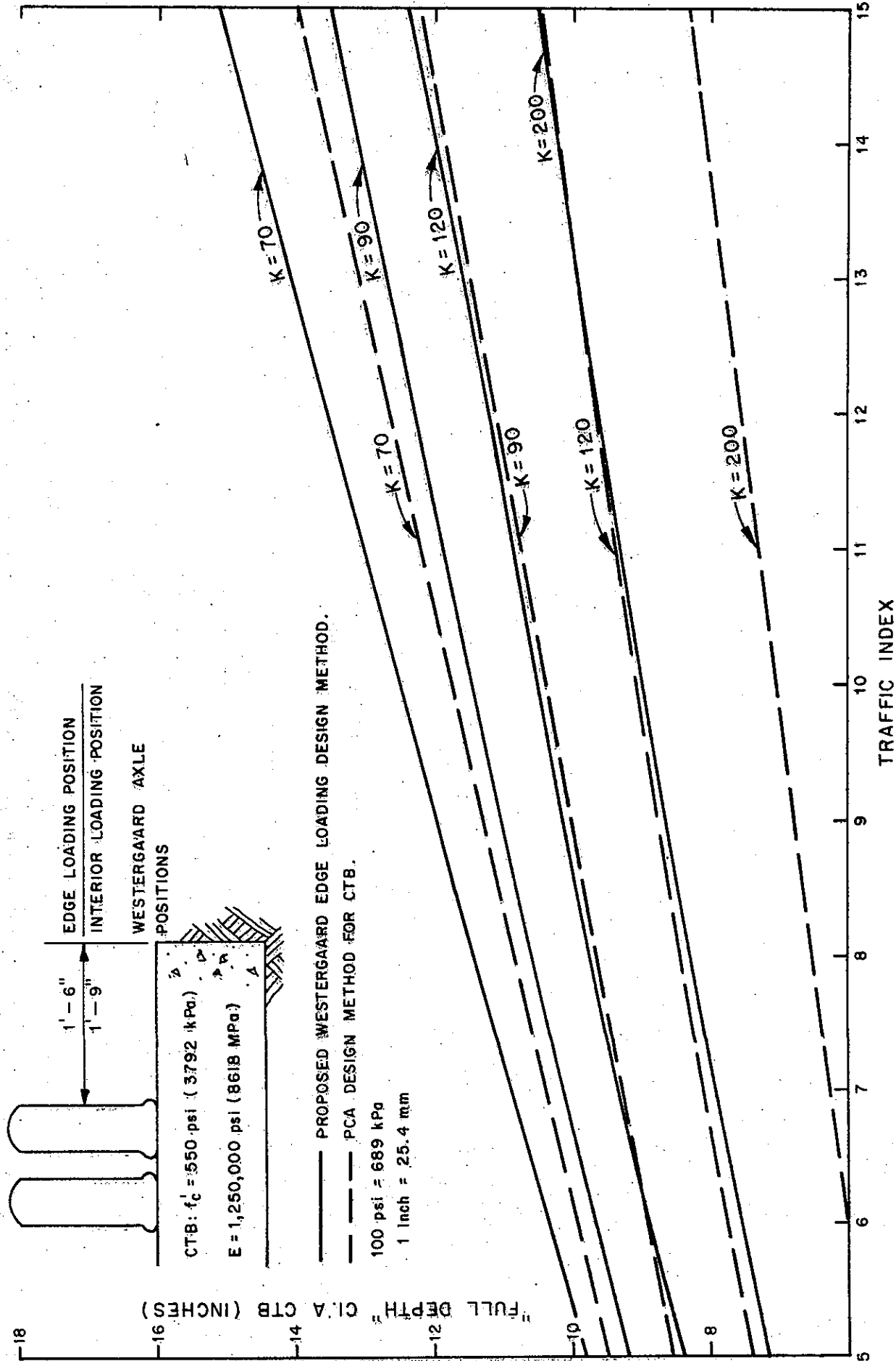


FIGURE 8



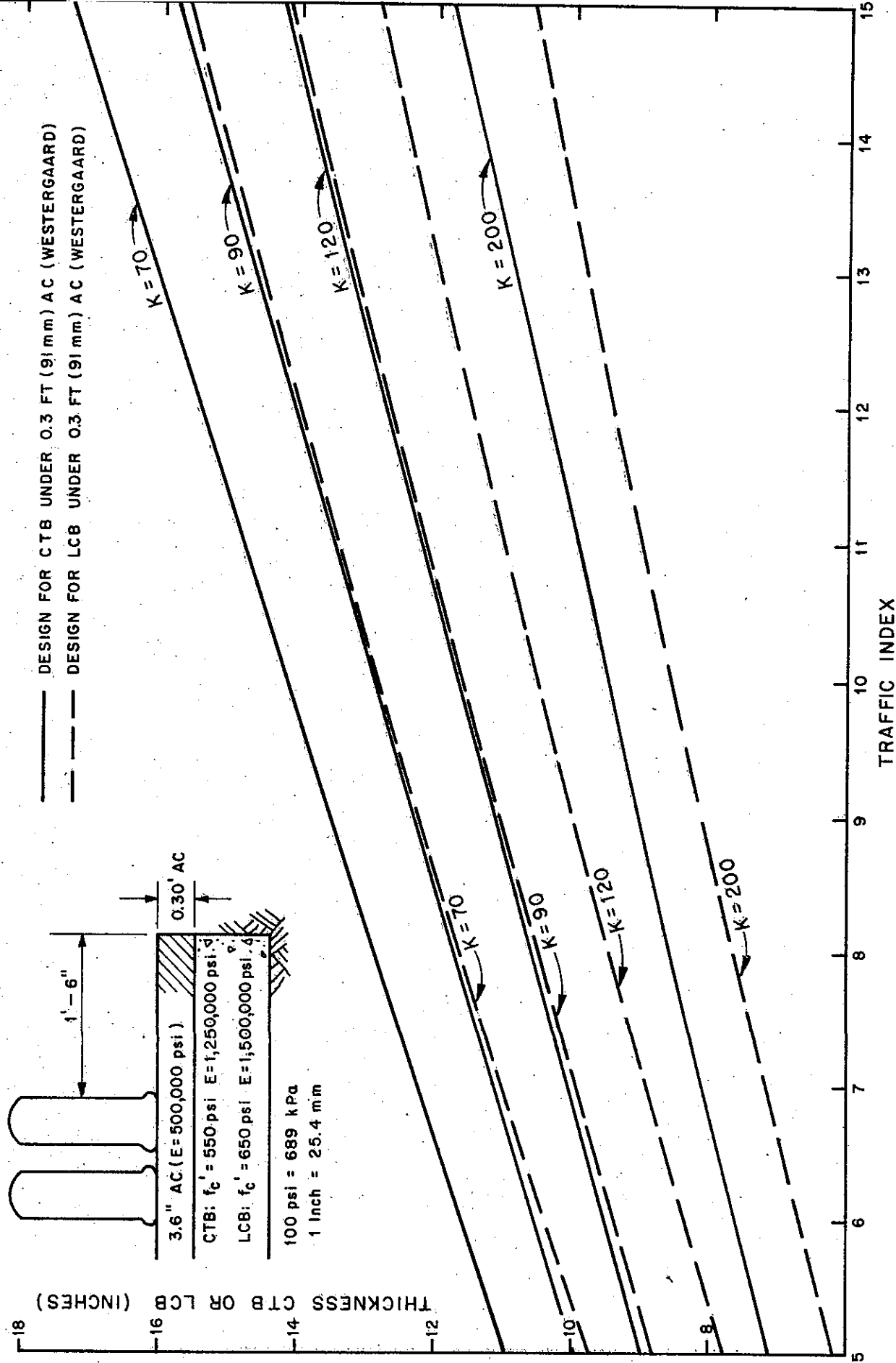
"FULL DEPTH" CTB DESIGN - PCA VS EDGE LOADING 1962 TRAFFIC

FIGURE 9

"Miner's rule", which is used for fatigue analyses by the Westergaard design method, may give unsafe values for fatigue analysis when heavier axle loads are considered. Since CTB and LCB are closely related to concrete, these findings should be considered. For this reason, the sensitivity of the Westergaard design method to increases in traffic loads may not be adequate. However, within the range of  $II$ 's included in Table 5, this sensitivity appears to be satisfactory.

A comparison of the thicknesses of CTB or LCB required by the Westergaard design method under a typical AC surfacing is shown in Figure 10. It can be seen that by the use of LCB, the thickness of the treated base can be reduced about 10% for the same traffic. Thus, it would appear to be desirable to use LCB instead of CTB.

The use of a constant "gravel factor"  $G_f$  for CTB or LCB for structural section design as is now required by the current Caltrans' method does not appear to be appropriate. Nussbaum and Larsen(38) state, "A comparison of load capacity based on equal deflections for soil cement and granular bases showed that the ratio of soil cement to granular base load capacity was about 1.5 for a 4 inch (102 mm) thickness and about 3.3 for a 10 inch (254 mm) thickness." In addition, the use of a constant  $G_f$  regardless of strength is another possible reason for the apparently conservative results provided by the Caltrans procedure. It is entirely possible that the strength of much of the CTB included in the structural section described in Table 5 was considerably in excess of the minimum values assumed for design purposes. The Westergaard design method, which consists of a fatigue design method which is stress related and can accommodate the strength and stiffnesses provided by the structural materials actually being used, seems to be preferable. This theoretical method may also be applicable to lime treated base if the flexural strength, stiffness, and flexural fatigue response, based on stress level, can be predicted with reasonable accuracy.



TRAFFIC INDEX VS BASE THICKNESS

FIGURE 10

Thus, for pavements composed of AC over CTB or LCB, the Westergaard method of analysis for edge loading, based on flexural strength and stiffness of CTB and stiffness of AC using predicted axle loads in conjunction with Miner's rule to determine fatigue response, appears to be an improvement over the present design method. The Westergaard method provides a structural section design that gives layer thicknesses that agree better with pavement performance than the method now used. The present method is generally more conservative for the pavement patterns of the 1960's. While the Westergaard method is less conservative for the past types of loadings than the present method, it agrees better with pavement performance. For the present and future considerably heavier loadings, the Westergaard method will provide pavement designs that are in reasonable agreement with test loading data from field, laboratory, and analytical research.

#### C. Rigid Pavement Design

There are several theoretical procedures that have been utilized to analyze rigid pavement for purposes of design. The first one extensively used was that by Westergaard incorporating the theory of elasticity. This method was later expanded by the Portland Cement Association (PCA) and is currently used by California as well as many other states. Burmister's elastic layer method that is sometimes used for flexible pavement design is not used extensively for concrete pavement design since it considers the pavement layers to extend horizontally an infinite distance and has no way of directly considering edge or corner loading which has proven to be critical for rigid pavement design.

The finite difference method, as introduced by Hudson and Matlock(39), has been utilized by the Texas Highway Department to some extent. This method involves using differential equations to determine traffic-induced deflections, stresses, and strains in the concrete for given structural sections. The finite element method requires

similar assumptions and is another theoretical analysis that has been used to some extent in research at the Universities of California, Kentucky, and Illinois with regard to rigid pavement. Both of these methods are more complex than the Westergaard design method or the PCA method.

The aforementioned analytic methods are based on controlling the concrete stress - the object being to keep the stress in the concrete slab below a certain level to prevent crack propagation. However, not all cracking is considered preventable (joint cracks, cracks caused by thermal contraction and drying shrinkage, and stress generated by uneven base support). Though cracking due to over-stressing or fatigue is very undesirable, it is not the only state that should be considered when designing a rigid pavement system. Three possible manifestations of pavement distress that must be considered are rupture, distortion, and disintegration. These manifestations are functions of load, environment, construction, maintenance, and time.

The AASHO Road Test conducted in Ottawa, Illinois, between 1958 and 1960 was the most comprehensive effort made to date to evaluate pavement under repetitive loading. A semi-empirical method of design for concrete pavement was developed from AASHO test data that is used as the main basis for concrete pavement design in Texas, Illinois, and many other states. The AASHTO method and the PCA method are generally considered to be the two best practical methods available for design of rigid highway pavements.

The following are the main inputs for the PCA method of design. The concept of concrete failing due to fatigue is fundamental.

1. The subgrade is characterized by "modulus of subgrade reaction", or "k" value, based on deflection under a plate load. An increase in this "k" value due to treating the subbase is calculated using correlations which are based on Burmister's analysis of a two-layer system.

2. Traffic is projected with the help of standard charts using design life and a yearly rate of traffic growth. (Unfortunately this design input often varies considerably from subsequent actual traffic.)
3. Different load safety factors (LSF's) are used for various types of facilities. These LSF's help account for various support conditions, weather, etc.
4. The maximum stress, assumed to occur when the load is at the joints or edge, is determined from charts developed for both single and tandem axle loads. These charts are prepared from influence charts developed by Pickett and Ray assuming no load transfer at the joint.
5. The fatigue life "used" by each load group is computed and these values then summed up to ensure that the total fatigue resistance of the pavement is not exceeded during the design life of the pavement. This method of fatigue summation for the various loads is called the "Miner rule".

The advantages of this PCA method are as follows:

1. The stresses under load can be calculated with reasonable accuracy and these stresses then used as a basis for design.
2. The effect of loads of varying magnitudes at various locations, i.e., edge loading or interior loading, can be considered.
3. Pavements designed by this method usually perform adequately in terms of loads carried versus calculated load carrying capacity.

The disadvantages of the PCA method as determined by experience in California and indicated by laboratory test results are as follows:

1. Unbalanced design - In California the outside (truck) lanes generally are the first to require maintenance while the passing lanes usually far outlast their design lifetime. This indicates that the passing lanes may be too conservatively designed relative to truck lanes.
2. Unrealistic Load Safety Factors - Hilsdorf and Kesler(37) determined that, based on laboratory tests, the PCA approach provides very conservative results for light loads and unrealistically long calculated fatigue lives when heavy loads are considered. Qualitatively, these trends seem to be confirmed by pavement performance in California. Even an occasional overload can "use up" a disproportional amount of fatigue life. The use of different load safety factors for passing and truck lanes is apparently not completely adequate to compensate for this difference in traffic density.
3. No direct consideration is given to the stress caused by temperature variation (warping or curling).
4. Though k-value appears to have a relatively small effect on design thickness, there is no recognition given to the actual strength or stiffness of CTB when determining the k-value of the base. Also, no way is provided for determining the stress in the CTB or LCB under the PCC.
5. The initial failure of the pavement in most cases consists of faulting near the joint due to non-uniform support and/or pumping. The design method has no way of directly considering this effect or its cause.

The main advantage of the AASHTO design method is that it is based on the performance of actual PCC pavements under load as observed during the most comprehensive study of "in-service" pavement that has been attempted. The range of loadings used for the test roads included enough variation to produce results that could be used to determine effects of single axle loading up to 40,000 pounds and tandem axle loading up to 48,000 pounds. However, the AASHTO procedure also has some disadvantages, at least with regard to California conditions.

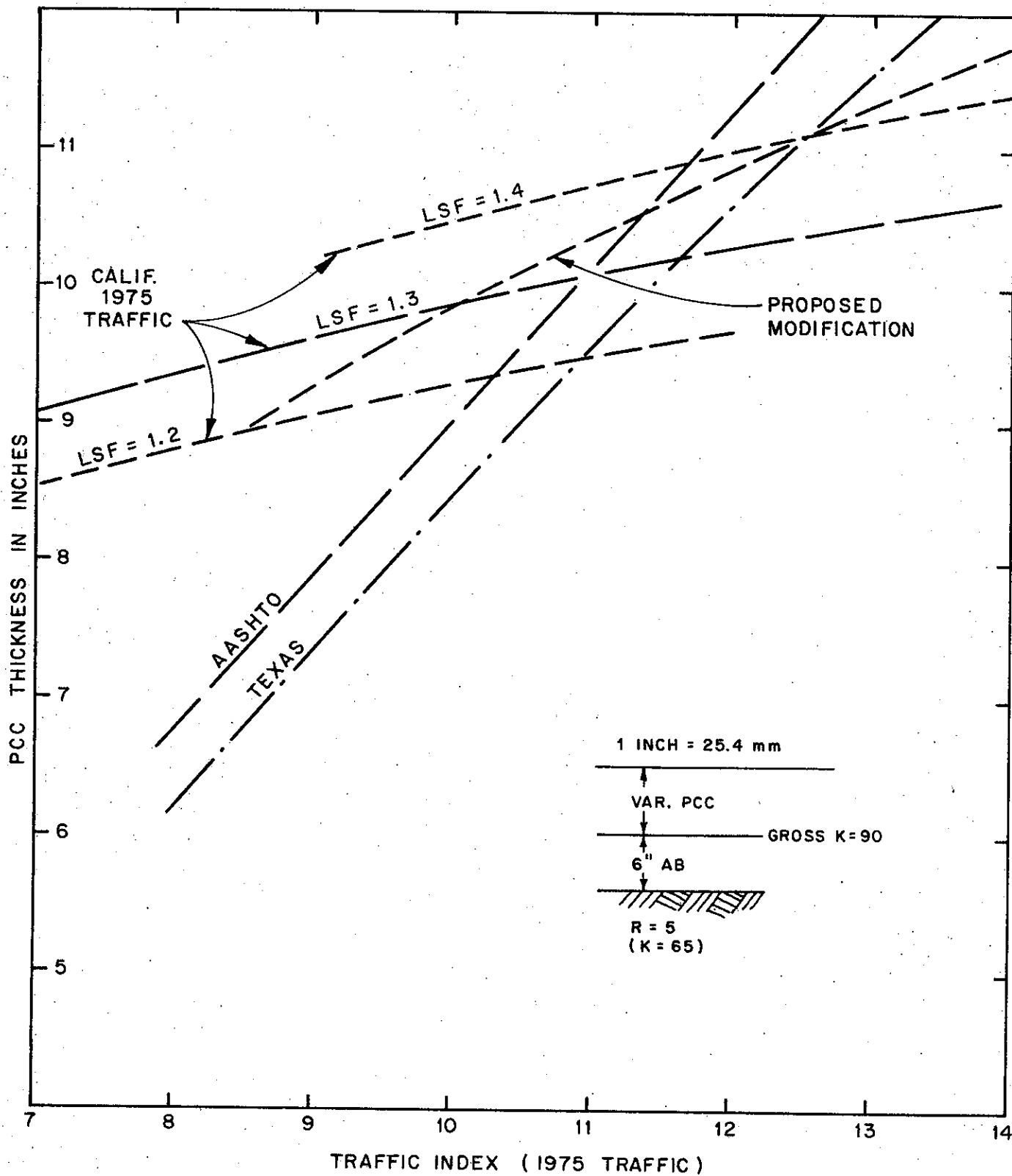
The subgrade for all the concrete pavements tested as part of the AASHO effort had an average CBR of between 1.0 and 1.6, which is equivalent to an R-value of about 5. Cement stabilized bases were used only in conjunction with AC so the performance of PCC supported by CTB was not evaluated. In addition, the freezing and thawing of the bases and subgrades that took place is not typical of most areas of California. The duration of the test was only 2 years, compared to the 20 year design life commonly used. The shorter 2 year test is probably more severe with regard to damage because the rest periods between truck loadings, which are known to influence fatigue life, were only about 1 minute whereas the rest periods would tend to be much greater for a typical roadway. Also, under actual service conditions, the average strength of the pavement under longer service would be higher. On the other hand, damage due to environmental conditions (moisture and temperature) could be progressive and, therefore, more severe for the longer lifetime, but the overall advantage would appear to be with the longer service life if the design of the pavement is such that environmental conditions are not extreme.

A comparison was made for subgrade and base conditions similar to those prevailing in the AASHO road test for concrete pavements. The AASHO road test employed six different thicknesses of concrete (3-1/2" (89 mm), 5" (127 mm), 6-1/2" (152 mm), 9-1/2" (241 mm),

11" (279 mm) and 12-1/2" (318 mm)) over aggregate subbases of 3 (76 mm), 6 (152 mm), or 9 inches (228 mm). Figures 11 and 12 show comparisons of the California method, using different load safety factors, with the Texas and AASHTO methods for conditions prevailing at the AASHO road test. This comparison indicates that for heavier traffic, both the AASHTO and Texas procedures dictate greater PCC thicknesses than does the Caltrans' procedure, particularly when the commonly used load safety factor (LSF) of 1.3 is used in conjunction with the Caltrans' procedure.

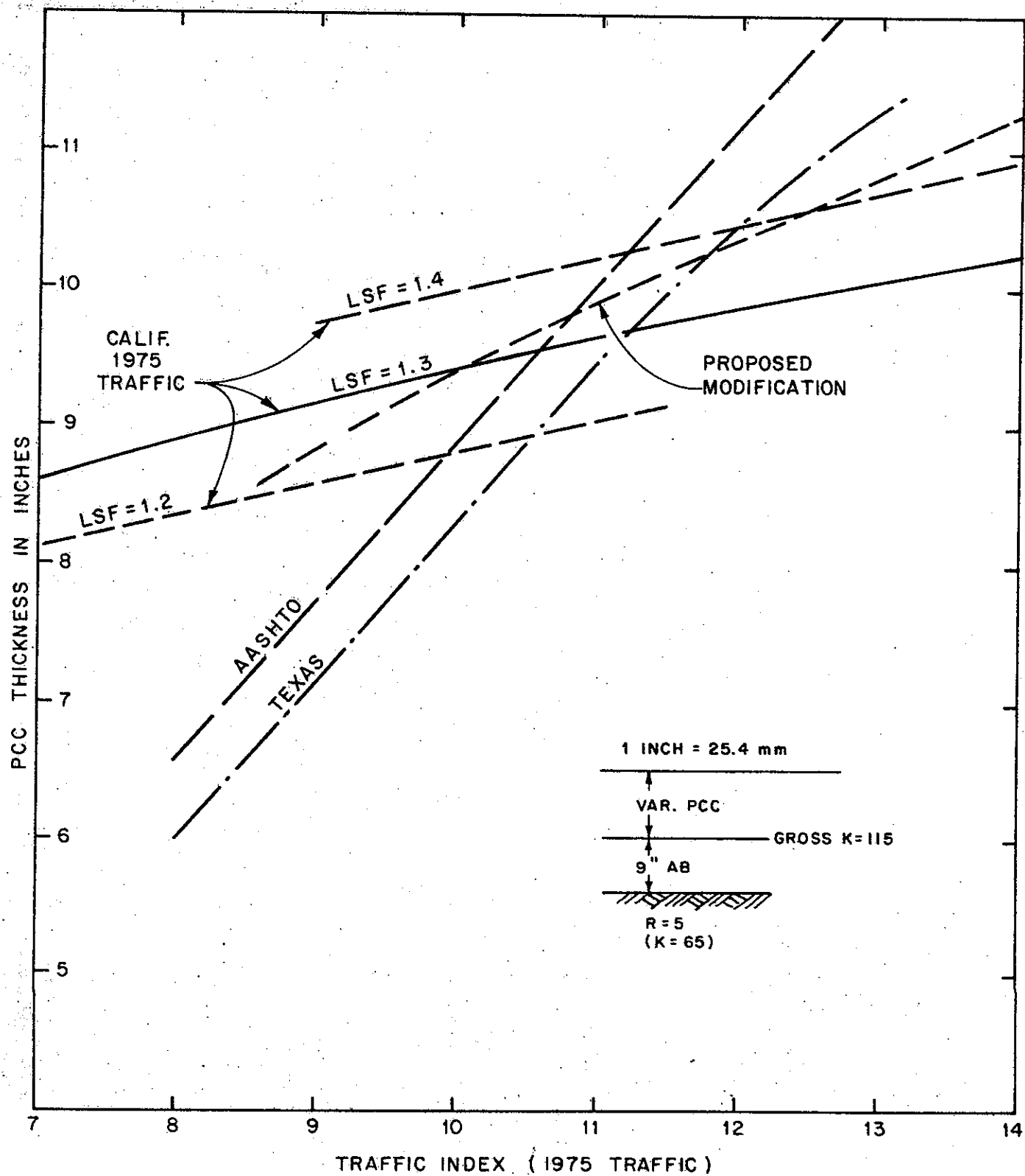
Using a LSF of 1.4 gives results in close agreement with the Texas design method for a TI of 12.5. Figure 13 shows a design comparison for conditions similar to those prevailing in California and indicates agreement with the California design method for a load safety factor of 1.3 at a TI of 10. Both the AASHTO and the Texas procedures dictate greater thicknesses than the Caltrans' method which normally employs a LSF of 1.3 for the truck lanes of Interstate highways and other multilane highways with considerable truck traffic. This, in conjunction with the concentration of distress in the truck lanes, indicates that the load safety factor should be greater than the one now used for the truck lane. A possible modification wherein the LSF increases with TI is shown in Figure 14. The effect of this proposed modification is indicated by a dashed line on Figures 11, 12, and 13. The proposed modification should tend to make the present California design method provide more equal pavement lifetimes for truck and passing lanes. It will provide pavement thicknesses in the truck lanes that, although greater than those now provided, will not be excessive in that they would still be less than those dictated by the Texas and AASHTO procedures for heavily traveled highways.

The cement stabilized subbase used in tests by PCA to determine k-value adjustment for CTB had a lower compressive strength but about the same stiffness as typical California CTB. For this reason, the subbase characterization used in the PCA concrete



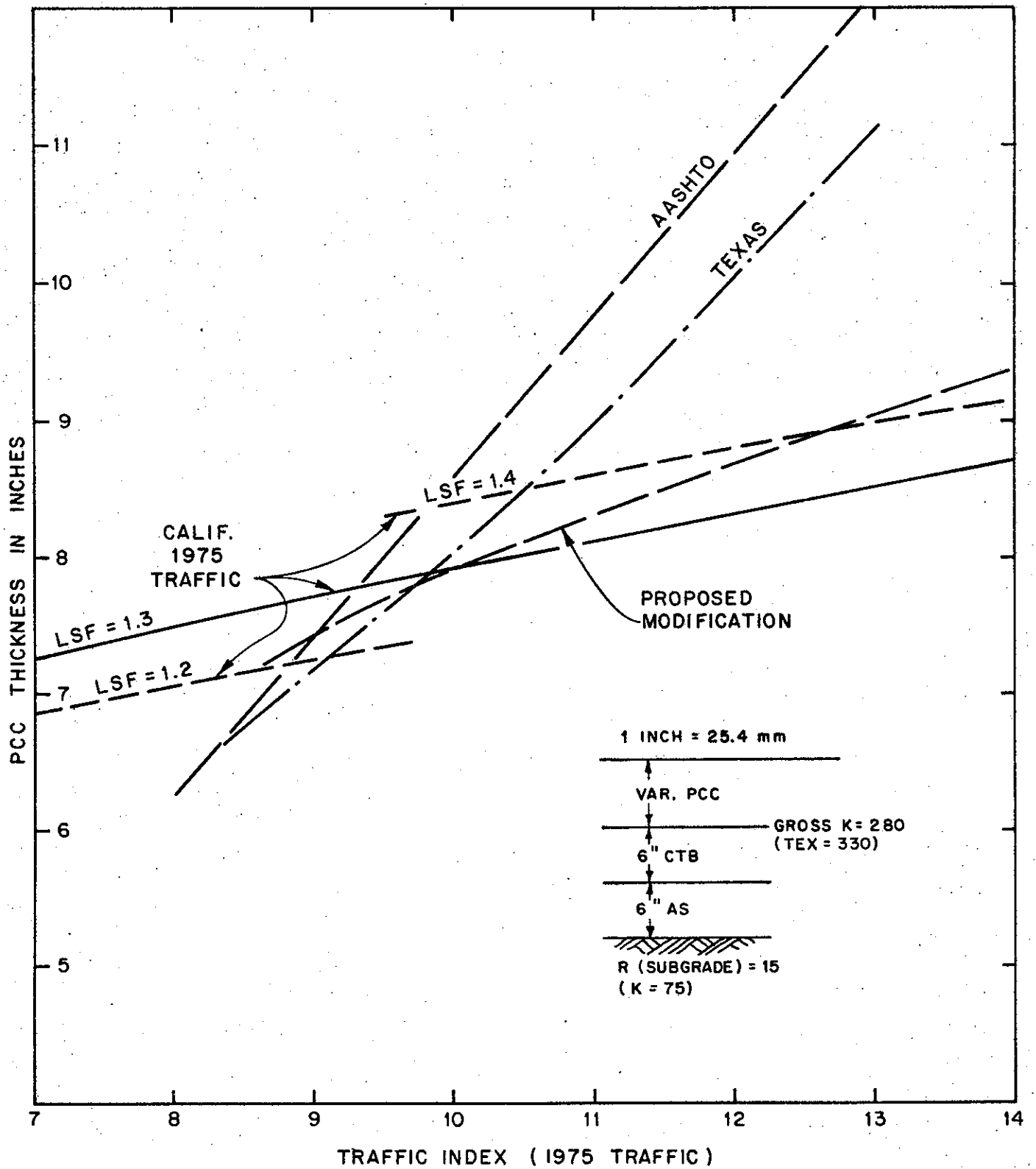
TRAFFIC INDEX VS PCC THICKNESS

FIGURE 11



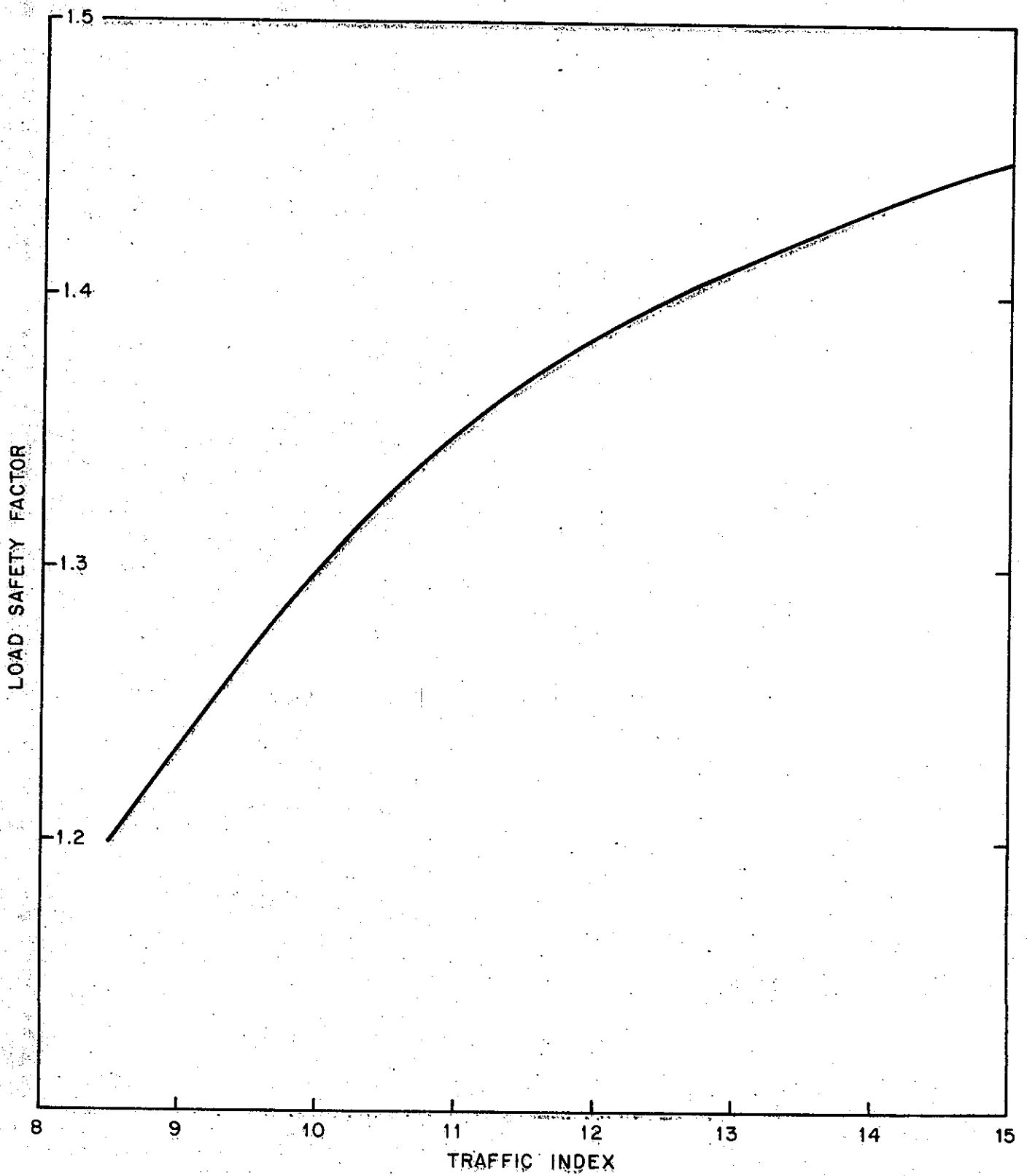
TRAFFIC INDEX VS PCC THICKNESS

FIGURE 12



TRAFFIC INDEX VS PCC THICKNESS

FIGURE 13



LOAD SAFETY FACTOR VS TRAFFIC INDEX

FIGURE 14

pavement design method would appear to be applicable to California conditions. If lean concrete base is used instead of CTB, the comparable thickness of the LCB could be reduced 10% according to the comparisons of the structural qualities of CTB and LCB. This lean concrete base has a strength and stiffness comparable to the cement treated subbase used in tests by PCA to develop a k-value adjustment chart for CTB under PCC airport pavement(40). This chart indicates equivalent k-values for thicknesses of CTB about 25% less than the PCA chart for highway pavements. For this reason, the 10% difference in required thickness would appear conservative provided the strength of LCB is in the range assumed.

There is presently no method in common use for determining stresses or strains in the CTB or LCB under PCC. If the critical stress conditions for the CTB are the same as those for PCC pavement, a modification of the Westergaard method could be used. There is a probability that in some cases the critical stress in both concrete and CTB occurs under edge loading about half way between joints. The PCC pavement cracking patterns in the AASHO test(41) indicates this possibility. Huang and Wang(30), in studies based on the finite element method, determined this to be the case for concrete pavement, as did Darter, Barenberg and Tabatabaie(32). Because of the one foot extension of the CTB or LCB beyond the edge of the PCC pavement that is customary in California, the stress determination for this case would be difficult, but the finite element method may be a possible mode of investigation. If the suggested modification of the LSF does not provide a more balanced design, it may become necessary to modify the pavement analysis to include an examination of the stress and fatigue resistance of the structural section considering an appropriate degree of composite action between the PCCP and the CTB or LCB.

The design service period of 20 years now being used in California should be reviewed. There is a strong feeling in Texas(42) that 20 years is an inadequate design period, especially for urban

freeways. Highway designers in England(43) also feel 20 years to be too short a service life for concrete pavements, because of greater problems involved in repair as compared to AC. In urban areas, it is unlikely that the current alignment will be replaced for periods well in excess of 20 years. Also, disruptions to traffic are more costly. For these and other reasons, it seems highly desirable to design for longer maintenance-free life.

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